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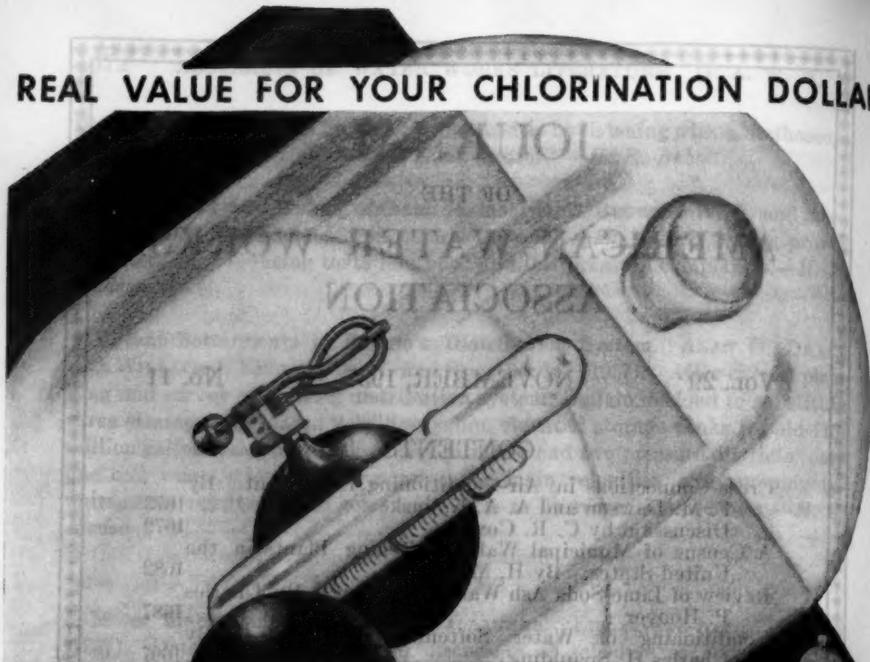

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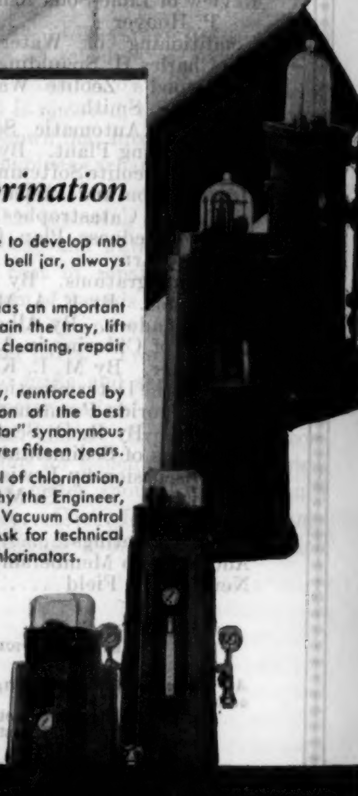

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OF THE

# AMERICAN WATER WORKS ASSOCIATION

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## CROSS-CONNECTIONS IN AIR-CONDITIONING EQUIPMENT*

BY F. M. DAWSON[1] AND A. A. KALINSKE[2]

The present intensive use of all types of air-conditioning equipment should be of great interest and concern to all waterworks engineers. Practically all such equipment uses water, thus making it necessary to connect certain parts of the apparatus either to a public or a private water supply. Two problems of major importance have been created: (1) the erratic and exceedingly high peak water demands during certain periods, and (2) the presence, both actual and potential, of cross-connection hazards which endanger the purity of the water supply.

This paper is concerned with the second of the above mentioned problems; namely, that of analyzing the dangers presented by the connection of the water supply system to air-conditioning equipment. These connections may and frequently do create conditions inimical to the sanitary quality of the water. To facilitate the detection of such cross-connections by inspectors, the various points where such connections may exist will be described.

Where city water is used directly as a cooling agent for condensing refrigerants in air-conditioning apparatus and then wasted into the

* Presented at the Buffalo Convention, June 10, 1937.

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[2] Instructor and Plumbing Research Engineer, State University of Iowa, Iowa City, Ia.

city sewers, the greatly increased use of water during periods when air-conditioning equipment is in use, severely taxes the waterworks plant to its maximum capacity. Furthermore, the sewers may become grossly overloaded in the more built-up portions of a city. Chicago may be cited as one city where this problem is acute. Obviously, with the increased use of air-conditioning equipment, the water departments of all cities will have to make definite regulations relating to water used for such purposes. These regulations will probably resolve into stringent rules relating to the use of water and extra charges for the peak demands. In order to make such rules effective the water supply must be metered.

### DIRECT CONNECTIONS TO CONDENSERS

The main use of water in air-conditioning apparatus is for condensing the refrigerant used in the cooling arrangement. In many instances, a direct continuous (i.e., not open) connection is made from the public water supply to the condenser. There are some condensers, known as the atmospheric type, in which the water discharges into an open trough and flows down the pipe coils of the condenser, which are exposed to the air. In such a case, there is no direct connection between the water surrounding the piping containing the refrigerant and the public supply, and no possibility of feeding back to the pure water mains.

Disregarding germicidal pollution for the moment, the factor which determines whether any such direct connection to condensers is to be permitted at all is the toxicity of the refrigerant. It is the writers' opinion that no direct connection between the public water supply and a condenser containing a toxic refrigerant should be permitted. Refrigerants such as ammonia, methyl chloride, sulfur dioxide, dichloroethylene, propane, and probably others must be considered as toxic under certain conditions. It is recognized that certain refrigerants with the proper dilution might have beneficial germicidal properties. However, no guarantee can be given that the dilutions might not become dangerous for human consumption.

Direct connections to non-toxic refrigerant condensers are not desirable for several reasons, one being the possibility of a direct connection being put in between the condenser waste and a sewer.

In place of the direct connection either a gravity tank supply or a surge tank and pump could be installed. In either case, the tanks should always be supplied with city water through an inlet located

above the overflow, or preferably, above the top of the tank. A type of float control valve should be used with the inlet above the sides of the tank and the float itself in the tank. If a surge tank and pump is to be used, it should be located near the condenser, since there would not be any long runs of piping to which pure water fixtures might become connected. When either a separate gravity feed tank system or a surge tank and pump system is installed, one fundamental principle governs; namely, that no pure water fixture connection should be made to this separate system. A tank with an open air gap type of inflow is installed for the purpose of breaking a cross-connection between the public water supply and the apparatus using water which is or may become unsafe. If a direct cross-connection exists between a separate system, which may contain unsafe water, and a pure water supply, even if the danger of contamination is remote, the water in the whole system cannot be considered as safe, although it may look as if it were. Obviously, if any separate system which might become contaminated is piped around to various points in a building, the chances of fixtures requiring pure water being connected to such systems becomes greater than if the piping is restricted. Therefore, the smaller the amount of separate supply piping, the safer and more ideal is the situation.

Everything that has been said about water pipe connections to condensers applies equally to cooling water connections to gas compressors. No direct connections should be allowed if toxic gases are being compressed, and preferably, none for any type of gas.

## WASTE WATER AND OVERFLOW CONNECTIONS

Frequently the waste water from a condenser is run directly to a sewer or drain through a continuous connection. If the condenser is directly connected to the water supply, there then exists a direct connection between the water pipes and the sewers. This is probably the worst type of cross-connection to be found in air-conditioning equipment. Such waste waters should discharge through an atmospheric gap of several inches (four inches recommended) before it enters the drain trap or other part of the sewer system. The gap is measured from the lower end of the discharge pipe to the top of the trap or fixture into which the waste water discharges.

It should also be mentioned that overflows from gravity feed tanks and surge tanks should not connect directly to any part of the drainage system. A broken connection is recommended. Long

overflow pipes in buildings are conducive to the creation of unsanitary conditions because of the opportunity always present for someone to use the overflow pipe as a waste from plumbing fixtures, either intentionally or not. All overflow pipes from tanks should be treated as if they carried impure water. In addition, they should be sufficiently large to reduce the possibility of clogging to a minimum.

### SUBMERGED INLETS IN COLLECTING TANKS

In two types of air-conditioning apparatus, open collecting pans or tanks are used. The water in these pans or tanks has to be considered impure even when it appears to be safe. This is particularly true when the condenser cooling water is re-used by cooling it after it leaves the condensers by means of ponds, sprays, or towers. Since the cooling of this water is accomplished primarily by evaporation, the supply must be replenished, which necessitates a water connection to the cooling pond or collecting pan at the bottom of the cooling tower. The water inlet to such ponds or pans must be above the maximum possible water level. It is also good practice to have the overflows from such ponds or tanks connected indirectly to the sewer system through an air gap.

Collecting pans are also used in air-washing and dehumidifying equipment. The water is forced by means of a circulating pump through nozzles, thus forming a spray. The air is drawn by fans or blowers through this spray. The water drops to a collecting pan and is re-used. There is usually a fresh water connection to the collecting pan for purposes of supplying make-up water and new water to be added after the pans and filters are cleaned. Obviously, the water in these collecting pans is unfit for domestic use, and therefore, all connections to such pans must be made above the maximum possible water level. Overflows must not be connected directly to the drainage system. Sometimes a centrifugal pump is used to circulate the cooling water and a city water connection is made to prime the pump. It is better practice to have all pump priming connections broken and an open funnel arrangement used.

### AUXILIARY WATER SUPPLIES

If the city water is not of the right temperature, is expensive, or has undesirable chemical properties, an auxiliary water supply is sometimes brought into a building for the air-conditioning equipment. It may be a private well water or river water. Experience has clearly shown that the presence of many such auxiliary supplies

makes the problem of elimination of direct cross-connections very difficult. Obviously, the cross-connection of such auxiliary supplies having a low sanitary index with the city water should be entirely forbidden. As far as air-conditioning is concerned, if such an auxiliary supply is below the sanitary standard of the city water, no provision should ever be made for connecting the city water directly to the piping system containing the unsafe auxiliary supply. The only safe provision that can be made for the use of the city water in the auxiliary system is by the use of a surge tank and pump. The city water would then discharge above the maximum water level of the surge tank and be used to supply water when the auxiliary supply fails. Clearly, even under the best of conditions, the presence in a building of an impure auxiliary water supply under pressure is an undesirable potential hazard.

If the auxiliary supply has a sanitary index equal to or better than that of the city water, then the regulations relating to provisions for connecting the city water to such a piping system might be more lenient. However, under no circumstances must a direct cross-connection be permanently installed, that is, embodied in the piping which is covered over or concealed. Offhand, it might not be apparent why a permanent cross-connection should not be allowed always between the city water and an auxiliary supply whose sanitary quality is better than that of the city water. The main reason is that if such an auxiliary supply is used, it leads to carelessness regarding the presence of cross-connections and various submerged inlets, since the auxiliary piping system may not be itself connected to any fixtures requiring pure water. Therefore, permitting a direct cross-connection between the city water and the auxiliary supply might be similar to allowing sewer cross-connections and submerged inlets from impure water sources to exist on the city water supply system itself. It is thus apparent that there is only one condition which would permit the permanent existence of a direct cross-connection between the city water and an auxiliary supply; namely, when the auxiliary water supply is at all times of equal or better sanitary quality than the city water, and if no cross-connections or submerged inlets of whatever sort exist on the auxiliary piping system.

## RE-USE OF COOLING WATER IN THE DOMESTIC SUPPLY SYSTEM

Cooling water from condensers and gas compressors is sometimes used in the domestic supply system in the larger buildings of some cities. Quite obviously such a practice should never be tolerated if

the cooling water comes from condensers or compressors handling toxic refrigerants or gases. By re-use in the domestic supply system is meant the piping of such water throughout a building to various plumbing fixtures. Whether or not drinking water fixtures are connected to such a system is immaterial. It must be remembered that the intentional or unintentional connection of such drinking water fixtures to this supply system will probably never be detected or ever corrected by ordinary inspections. We must always be alert for the presence of potential hazards in addition to the actual ones.

Considering the great expense incurred and care taken to provide and maintain a pure water supply, the use of such cooling water in the domestic supply system, even if coming from condensers and compressors handling non-toxic materials, is far from desirable. It is something that probably might be tolerated temporarily under certain conditions, but should not be looked upon or promoted as a safe and permanent procedure.

Sometimes the water from these condensers and compressors discharges and flows by gravity through a considerable length of piping to a surge tank, from which the house pumps obtain the water for the domestic supply. Here again another significant potential danger looms up. Pipes carrying water under gravity-flow conditions are ordinarily considered as waste lines; therefore, someone not very familiar with the piping system in a building might easily connect the waste from a plumbing fixture to such a water line. Pollution of the entire water supply system would result.

## RETURN OF COOLING WATER TO CITY MAINS

Recently a plan was suggested to conserve city water by pumping the cooling water from condensers back into the street water mains. Certainly anyone having a knowledge of the work of the waterworks engineers and the departments of health in providing and maintaining a pure water supply, and of the enormous effort that must be put forth to isolate and eliminate undesirable connections from the maze of piping in our large buildings, should not suggest such a scheme. The potential hazards that would be introduced under this plan would be numerous and frequent. A slightly faulty or wrong connection would pollute the water supply for an entire district. It is not a plan worthy of the ingenuity and advances made in recent years by engineers in the air-conditioning industry.

## SUMMARY AND CONCLUSIONS

From the foregoing discussion it may be concluded that the correct observance of a few fundamental principles in regard to design and installation of air-conditioning equipment will eliminate practically all water-pollution hazards usually found in connection with such equipment. Briefly stated they are:

(1) *Direct, or continuous, water connections from pure water mains to condensers or gas compressors for cooling purposes should not be made* particularly if toxic refrigerants or gases are handled. The connection should always be broken by allowing city water to enter a gravity feed tank or pump surge tank above the highest water level in the tank. The surge tank with pump located near the point of water use is desirable.

(2) *The waste from a condenser or compressor should never be directly connected to any part of the drainage system.* A clear air-gap of about four inches should exist. Overflows from gravity feed tanks, surge tanks, or various collecting pans should not be directly connected to the drainage system.

(3) *Water-supply inlets to collecting pans in air-washers, de-humidifiers and to cooling ponds or towers must be above the highest possible water level.*

(4) *No direct cross-connection*, or provision therefor, *should be allowed between the city water and an auxiliary supply* from a private source unless such auxiliary supply has a sanitary quality equal to or better than the city water, and then only when no cross-connections or submerged inlets of whatever sort exist on the piping system using the auxiliary supply.

(5) *Re-use of condensing and cooling water in the domestic supply system is not desirable and should be prohibited* if such water comes from condensers or compressors handling toxic materials.

(6) Provisions should be made for adequate inspection.

(7) Last, but most important of all, *the water and waste piping connections to air-conditioning equipment should be made by skilled and licensed men* who understand the health hazards of cross-connections and appreciate the action of back-siphonage.

*Discussion by* C. R. Cox.[1] The marked increase in the number of air conditioning units has led to considerable concern on the part

[1] Chief, Bureau of Water Supply, Division of Sanitation, State Health Department, Albany, N. Y.

of water supply officials as to their ability to supply the required volume of water. While this concern is well justified, certain secondary problems of public health importance should not be overlooked.

Any marked increase in consumption of water, especially during the summer season when dry weather is to be expected, is likely to make necessary the use of emergency sources of supply, many of which may be far from satisfactory from a public health viewpoint. Furthermore, this increased demand for water may overtax existing treatment plants and lead to a reduction in the effectiveness of treatment. Under extreme conditions, the demand for water in taller buildings may lead to negative pressures and consequent back-siphonage from plumbing fixtures with its adherent public health menace. Finally, the need for large volumes of water for cooling purposes is leading to the development of auxiliary sources of water supply on private property. Many of these are exposed to pollution which is of significance when such supplies are cross-connected to municipal systems. It is evident, therefore, that public health and water supply officials must give careful consideration to these problems.

Recent discussions have indicated a feeling on the part of many water supply officials that the health officials should be primarily concerned with the elimination of any public health menace resulting from the use of air conditioning equipment, whereas others feel that water supply officials must take the initiative. Experiences in New York State in the control of cross-connections between municipal and auxiliary water supply systems, and interconnections between water supply and drainage systems through plumbing fixtures, have definitely indicated the joint responsibility of various authorities.

Fundamentally, of course, the property owner is responsible in seeing that air conditioning equipment, etc., is so installed and operated as to prevent any public health menace. Unfortunately, however, the average property owner is not familiar with the technical problems involved and thus an educational program is needed. At the present time, however, it seems possible to conclude that jurisdiction of such matters should be as follows:

*Property Owner.* Ultimately responsible for conditions on his property and in the employment of competent firms to install air conditioning and similar equipment.

*Manufacturers.* Manufacturers are responsible for the design and installation of their equipment so as to minimize the consumption of

water and prevent menace to public health. Fortunately there is a trend towards the design of equipment requiring less water than previously was the case.

*Local Water Supply Authorities.* Municipal laws and plumbing codes including building codes, etc., should be so enforced as to protect the interests of the public and insure adequate water supply services to all properties. Water supply officials should formulate an administrative program in coöperation with departments of health and public works, usually enforcing plumbing and building codes respectively, otherwise unified control will not be secured. It seems especially necessary for water supply officials to be authorized by ordinance to issue permits for specific installations of air conditioning equipment. No doubt many existing municipal ordinances will have to be revised to permit unity of action.

*Local Health Departments.* Local health departments of the larger municipalities usually employ sanitary engineers competent to supervise air conditioning equipment in the interests of public health, which ordinarily would form a portion of their enforcement of the local plumbing ordinances. In general, however, it would seem that the conservation of water would be fostered by having the water supply officials control the installation of air conditioning equipment as indicated above and for local health departments to enforce the plumbing code, insofar as necessary in the protection of health, and for them to coöperate with water officials when questions of public health are involved.

*State Health Departments.* Due to the extensive area supervised by state health departments, it is impossible for state sanitary engineers to exercise direct supervision of air conditioning equipment, etc. These departments, however, may be of great assistance in the promulgation of model plumbing codes for the guidance of municipal officials; in conducting research in coöperation with universities and manufacturers; and, in educational activities intended to stimulate due consideration of the problems involved.

# CENSUS OF MUNICIPAL WATER SOFTENING PLANTS IN THE UNITED STATES—OCTOBER 1, 1937

By H. M. Olson

Editor's Note.—The data herewith tabulated have been assembled by H. M. Olson of the Ohio Salt Co.,* Zanesville, Ohio.

Since the information contained is of broad and current interest to the entire water supply industry, permission to publish it has been requested by the Editor of this Journal and granted by its author.

Unless otherwise indicated, the data for each state were furnished by the Engineering Division of the Department of Health.

| STATE | ZEOLITE PROCESS | LIME AND SODA PROCESS | |
|---|---|---|---|
| Alabama | Cherokee<br>Courtland | None | |
| Arizona | Grand Canyon | Prescott<br>Yuma | |
| Arkansas | None | Clarksville<br>Wynne | |
| California | Contra Costa<br>Menlo Park<br>San Mateo | Beverly Hills<br>Coronado<br>San Mateo<br>Santa Barbara | |
| Colorado | None | None | |
| Connecticut | None | None | |
| Delaware | None | None | |
| Florida | Hollywood<br>Miamishores<br>Sarasota | Arcadia<br>Belle Glade<br>Boca Raton<br>Canal Point<br>Clewiston<br>Coca<br>Dania<br>Daytona Beach<br>Ft. Lauderdale<br>Ft. Meade<br>Hialeah<br>Homestead<br>Live Oak | Miami<br>North Miami<br>Opa Locke<br>Ormond<br>Panama City<br>Port Mayaca<br>Sea Breeze<br>St. Augustine<br>St. Petersburg<br>Tampa<br>Tarpon Springs<br>Vero Beach<br>Welaka |

* Associate Member, American Water Works Association.

| STATE | ZEOLITE PROCESS | LIME AND SODA PROCESS | |
|---|---|---|---|
| Georgia | Calhoun | Dublin | |
| | | Thomasville | |
| Idaho | None | None | |
| Illinois | Ashland | Anna | Nekoma |
| | Clarendon Hills | Bloomington | Newman |
| | Cobden | Bluffs | Okawville |
| | Colchester | Chandlerville | Peru |
| | Fisher | Decatur | Princeton |
| | Greenfield | Des Plaines | Quincy |
| | Hammond | Elgin | Springfield |
| | Homewood | Fairbury | Sullivan |
| | Jonesboro | Freeburg | Villa Grove |
| | Maywood | Hartford | Virginia |
| | Monticello | Hinsdale | Western Springs |
| | Roxana | Kankakee | Winchester |
| | Sparland | Lebanon | Woodstock |
| | Stonington | Mattoon | |
| | Zion | | |
| Indiana | Bicknell | Ft. Wayne | |
| | Crown Point | Lebanon | |
| | | Marion | |
| Iowa | Glidden | Adel | Indianola |
| | Laurens | Ames | Jefferson |
| | Lohrville | Belmond | Oskaloosa |
| | Mt. Vernon | Cedar Rapids | Ottumwa |
| | | Churdan | Perry |
| | | Eagle Grove | Postville |
| | | Farnhamville | Roland |
| | | Humboldt | Shenandoah |
| | | | Storm Lake |
| Kansas | None | Athens | Hoisington |
| | | Baxter Springs | Independence |
| | | Beloit | Iola |
| | | Clay Center | Lawrence |
| | | Clyde | Lincoln Center |
| | | Coffeyville | Linn |
| | | Council Grove | Manhattan |
| | | El Dorado | Neodesha |
| | | Emporia | Pittsburgh |
| | | Erie | Russell |
| | | Fort Leavenworth | Topeka |
| | | Hanover | Washington |
| Kentucky | Guthrie | Owensboro | |
| | | Russellville | |
| Louisiana | St. Joseph | Colfax | New Iberia |
| | | Houma | New Orleans |
| | | Lake Charles | Westwego |
| Maine | None | None | |

| STATE | ZEOLITE PROCESS | LIME AND SODA PROCESS | |
|---|---|---|---|
| Maryland | None | None | |
| Massachusetts | None | None | |
| Michigan | Coopersville | Ann Arbor | Midland |
| | East Lansing | Anvil Location | Rockford |
| | | Benton Harbor | Saginaw |
| | | Dundee | Sibley Quarry |
| | | Grand Rapids | Wayne |
| Minnesota | None | Little Falls | |
| | | Madison | |
| | | Minneapolis[1] | |
| Mississippi | None | None | |
| Missouri[2] | Marshall | Boonville | Mexico |
| | Oregon | Caruthersville | N. Kansas City |
| | Parkville | Crystal City | Princeton |
| | St. Genevieve | Excelsior Springs | Rockport |
| | | Jefferson City | St. Louis |
| | | Kirkwood | St. Louis Co. |
| | | Liberty | Sailsbury |
| | | Marceline | Weston |
| Montana | None | None | |
| Nebraska | None | None | |
| Nevada | None | None | |
| New Mexico | Gallup | None | |
| New Hampshire | None | None | |
| New Jersey | South Orange | None | |
| New York | Bethlehem | None | |
| | Dexter | | |
| | E. Aurora | | |
| | E. Rochester | | |
| | Elba | | |
| | Palatine Bridge | | |
| | Thornwood | | |
| | Utica | | |
| | Yorktown Heights | | |
| North Carolina[3] | Haw River | None | |
| | Valdese | | |
| North Dakota | None | Fargo | |
| | | Grand Forks | |
| | | Harvey | |
| | | Jamestown | |
| | | New Rockford | |
| Ohio | Berea | Ada | Canal Winchester |
| | Burton | Alger | Centerburg |
| | Cedarville | Baltimore | Centerville |
| | Cuyahoga Falls | Batavia | Cincinnati |
| | Elmore | Beach City | Columbus |
| | Findlay | Bellevue | Crooksville |
| | Genoa | Bloomdale | Defiance |

[1] Construction in progress

[2] Information on zeolite plants from manufacturers of equipment. Information on lime soda plants from A. W. W. A. 1931 Census—later collected data not available.

[3] Information furnished by N. D. Doane of Permutit Co.

| STATE | ZEOLITE PROCESS | LIME AND SODA PROCESS | |
|---|---|---|---|
| Ohio (*concl.*)— | Grove City | Delat | Mt. Sterling |
| | Lancaster | Delaware | Mt. Vernon |
| | Lowellville | Findlay | Mt. Victory |
| | Mechanicsburg | Fostoria | Newark |
| | N. Baltimore | Fremont | New Richmond |
| | New Bremen | Gallipolis | New Washington |
| | New London | Georgetown | Niles |
| | New Madison | Girard | Oak Hill |
| | New Philadelphia | Glendale | Oberlin |
| | Pemberville | Glouster | Piqua |
| | Plymouth | Grafton | Reading |
| | Rittman | Granville | Rio Grande |
| | Wapakoneta | Greenville | Roseville |
| | Worthington | Hamilton | Sandusky |
| | | Hebron | St. Clairsville |
| | | Hudson | Sunbury |
| | | Johnstown | Troy |
| | | Kent | Upper Sandusky |
| | | Kenton | Warren |
| | | Leesburgh | Waterville |
| | | Leroy | Westerville |
| | | Marion | Williamsburg |
| | | Marysville | Woodville |
| | | Massillon | Wooster |
| | | McDonald | Wyoming |
| | | Medina | Youngstown |
| | | Milford | |
| Oklahoma | None | Oklahoma City | |
| Oregon | None | None | |
| Pennsylvania[4] | Ambridge | Avella | Lowber |
| | Bellevue (McKees Rocks) | Braddock | Manhiem Twp. |
| | Coraopolis | Brave | Maxwell |
| | Edgeworth | Carrick | New Bethlehem |
| | Sewickley | Coalport | Oakmont |
| | Somerset | Cokeburg | Reading |
| | Springdale | Ford City | Richeyville |
| | West Kittanning | Grove City | Saltsburg |
| | Wyomissing | Homer City | Snowden |
| | | Hooversville | Tarentum |
| | | Jenners | Washington |
| | | Jonestown | Wendel |
| | | Lancaster | West Newton |
| | | Library | West Pittsburgh |
| Rhode Island | None | None | |

[4] Information compiled from a variety of sources.

| STATE | ZEOLITE PROCESS | LIME AND SODA PROCESS | |
|---|---|---|---|
| South Carolina[5] | Bamberg | Orangeburg | |
| | Sullivan Island | | |
| South Dakota | None | Aberdeen | Watertown |
| | | Elk Point | |
| Tennessee | None | None | |
| Texas | Belleville | Austin | |
| | Glidden | | |
| | Huntsville | | |
| | Three Rivers | | |
| | Trinidad | | |
| Utah | None | None | |
| Vermont | None | None | |
| Virginia | Bridgewater | Bristol | |
| | Dublin | | |
| Washington | None | None | |
| West Virginia | Borderland | Cornwell | Holden |
| | Morgantown | Grant Town | Mt. Hope |
| | Wellsburgh | | |
| Wisconsin | Galesville | Columbus | Platteville |
| | | Evansville | Waupun |
| | | Neenah | |
| Wyoming | None | Gebo | |
| | | North Cody | |

[5] Information furnished by N. D. Doane of Permutit Co.

Zeolite plants listed are those designed for softening the supply. Plants primarily intended for iron or manganese removal are not included.

Plants listed under the Lime-Soda heading include those which use lime, soda, lime and soda or any usual combination of other materials with lime or soda where the result attained is a material reduction in the hardness of the supply.

# REVIEW OF LIME-SODA WATER SOFTENING*

By Charles P. Hoover

(*Chemist, Filtration Plant, Columbus, Ohio*)

Mr. George W. Fuller read the first paper ever presented to this association on water softening for municipal supplies, at the 26th annual convention held in Boston in July 1906. At that time there were only two municipal water softening plants in North America. One was at Winnipeg, Canada, placed in operation in 1901, and the other at Oberlin, Ohio, which started operating in 1903.

Mr. C. Arthur Brown (present at this meeting) designed the Oberlin plant which was not only the first municipal water softening plant in the United States, but was the first plant in this country or abroad using both lime and soda ash for a municipal supply.

The water at Winnipeg was taken from wells and had a total hardness of about 486 parts per million. The permanent hardness was 148 parts per million, but was not removed. Lime softening reduced the temporary hardness to 80 parts per million leaving a hardness in the finished water of 228 parts per million, far too hard to be regarded as satisfactory.

Oberlin carried the softening treatment somewhat further, in that the hardness was reduced from about 300 parts per million to about 80 parts per million.

The earliest municipal plants of which there seem to be a record were placed in operation at Canterbury and Southampton, England. The Southampton plant was placed in operation in 1888 and the one at Canterbury at a slightly earlier date. The designers of the early municipal softening plants built in this country following the construction of the Oberlin plant, deserve especial recognition and praise for the splendid plants they built. They were pioneers and had few or no precedents to follow. It was necessary for them to design every piece of important equipment needed and many of the appurtenances incorporated into the plants. Design efforts now, as compared to then, are rather simple. There is now a wide choice of

* Presented at the Buffalo convention, June 8, 1937.

chemical feeders, lime slakers, mixing devices, gauges, and controllers.

When the Columbus, Ohio plant was built, all of these devices were developed and designed for the job. I have just had access to Mr. John H. Gregory's original notes on the design of the Columbus plant which was started in 1905. Many of the notes cover conferences with Mr. Geo. W. Fuller. I was amazed, in reading them, to find how few of the facts needed to execute the design of a water softening plant were known at that time. The early accomplishments, it seems to me, are worthy of special commendation. It is true there were some industrial plants. The aggregate capacity was estimated by Mr. Fuller in 1906 to be 50,000,000 gallons daily. The operation of these plants was apparently, however, not satisfactory or entirely successful. Mr. P. A. Maignen in his discussion of Mr. Fuller's paper, made the following statement:

> "The industrial systems of water softening, which have been introduced in this country during the last few years, have not proved reliable and there is not, perhaps, one single installation of the kind alluded to where the engineer, inventor or contractor in charge of the plant would consider the treated water fit to drink. Sometimes an excess of lime, sometimes an excess of soda are used, at other times the chemical reactions are not complete and the water is not softened, much therefore, has to be done in the way of improving the industrial water softening plants, as at present installed."

At the time Mr. Fuller presented his paper before this association in 1906, the Columbus, Ohio, water softening and purification plant, was under construction. This was to be the largest complete water softening plant in the world. It was approximately 10 times as large as Winnipeg and 180 times as large as Oberlin. At this time plans and specifications had also been completed for a softening plant (to use lime only) at New Orleans, and a little later a plant was built at McKeesport, Pa. St. Louis and a few other cities were adding lime in conjunction with sulphate of iron, more as a coagulant than for softening, yet reducing the hardness somewhat. It seemed therefore that the trend was toward water softening. Mr. Maignen further on in his discussion of Mr. Fuller's paper said:

"The speaker is very glad that the subject has been so ably and strongly presented to the society by Mr. Fuller. It will certainly mark the beginning of an epoch in the art of water purification in this country."

Mr. W. B. Gerrish, Superintendent, Water Works of Oberlin, Ohio, said in 1905:

"The possibilities of the lime-soda process for municipal use seems to be little appreciated as yet, but the time will doubtless come when it will be the means of solving some of the vexing problems of water supply in the middle west."

Some of the difficulties encountered and observed at Oberlin after 2 years of operation were recorded by Mr. Gerrish in a paper before the New England Water Works Association in 1905. He told of the tendency of softened water to form deposits, especially where there was a restriction and consequent rapid flow, and of the coating of sand grains with calcium and magnesium. The effective size of the sand was changed from 0.37 m. m. to 0.55 m. m. and the uniformity coefficient changed from 1.62 to 2.0. He also told of the drop in alkalinity between the filters and the center of the distribution system, which he construed as a danger signal since it implied a deposition in the pipes and meters. Mr. Gerrish reported that no appreciable after-precipitation or deposition of carbonate of lime had formed in the distribution system or meters after two years of operation. He intimated that there would probably be no incrustation troubles in the distribution system if sufficient time were allowed for the softening reactions to be completed, but added that if there should be trouble from this cause it could be easily remedied by putting $CO_2$ in the water to fix the carbonates. He cautioned against the enthusiasm on the part of manufacturers of softening machines which led them to make claims as to how quickly the water would pass through the process. He said that the process was one requiring considerable time and to attain the best results there should be about six days of sedimentation before filtration.

In discussing Mr. Gerrish's paper, Mr. Geo. C. Whipple, elaborated on the time element which enters into chemical reactions involved in the softening of water. He pointed out that while it is true that most of the precipitate settles in about six hours, several days are required

for the final reaction to be completed. He called attention to the fact that at Oberlin where the period of sedimentation was long, the alkalinity was often reduced to as low as 30 p.p.m.; (average was 57) whereas, at Winnipeg, where the time for chemical reaction was less than 2 hours, the resulting alkalinity was about 80 p.p.m. and that other plants, presumably industrial plants, where the settling period was intermediate between Oberlin and Winnipeg, intermediate results were obtained. Mr. Whipple also pointed out that the deposit which took place on the sand grains of the filter and in the distribution pipes might be prevented by recarbonation and concluded that if municipal water softening plants were to develop, as seemed likely, recarbonation should receive very careful attention. As I look back over the period of 29 years that I have been connected with the operation of the Columbus, Ohio plant, I am sometimes amazed and somewhat critical of the fact that it has taken so long to solve some of the pressing problems pointed out by these men more than 30 years ago. Even today the answers to some of the more important problems of this supposedly simple process have not yet been satisfactorily solved.

In the presentation of this paper, it will be possible to present in abstract only, the improvements and advancements that have been made.

## HANDLING AND APPLICATION OF CHEMICALS

Large quantities of chemicals can now be economically handled by unloading them from the cars by means of power shovels or by means of pneumatic conveying devices and they may be cheaply elevated into bins from which they are subsequently fed into the water by gravity.

Chemical feeding devices for weighing the chemicals instead of measuring them into the water have been highly perfected and have resulted in more uniform application of the softening chemicals. These same devices equipped with continuous lime slakers permit the elimination of solution tanks with all their attendant difficulties. The improvement in handling and applying chemicals eliminates the most laborious, dusty and disagreeable job about a water softening plant. Much improvement has also been made in devices for applying the chemicals automatically in direct proportion to the volume of water treated. The introduction of rotary kiln burned lime has also simplified lime slaking.

In the early days of water softening, the common practice was to add saturated lime solution to the water to be softened. Saturated lime water is difficult to prepare in large quantities and fortunately it has been found to be unnecessary and (so far as I know) is not used at any municipal plant today. The construction of early plants was somewhat involved because it was thought necessary to add lime first and then, after an adequate period of time, to add soda ash. This notion was founded on a false conception of the chemical reactions involved and is not now considered necessary.

### MIXING TANKS

There has been much written about mixing or agitating devices. Originally, baffled tanks with various combinations were employed. These were followed by the use of mechanical agitators, usually provided with rotating paddles or propellers; now the so called flocculator or super-mix is being adopted in almost all of the new plants. In this latest design paddles are attached to a horizontal shaft placed at right angles to the flow of water. These later devices insure uniform velocities throughout the mixing tanks and provide greater accessibility and flexibility over the original baffled tanks.

### SETTLING BASINS

Originally, settling tanks were designed for intermittent cleaning and were provided with bottoms having steep slopes to an outlet gutter or to outlet sumps distributed over the area of the floor. Now, at practically all the larger plants and at many of the smaller ones, equipment is provided for continuously removing the sludge. By this method the capacity of the basins is not decreased each successive day through the accumulation of sludge; periodic shut downs are not necessary; and, the muss and labor of frequent basin cleaning is eliminated.

### LIME SLUDGE

The disposal of lime sludge is one of the major problems in lime-soda ash municipal softening. The usual procedure is to discharge it into streams. This is satisfactory if the flow and velocity is sufficient to carry it away, otherwise it is apt to create an unsightly appearance on the banks and in the stream.

The Columbus plant is fortunate in having an abandoned quarry into which the sludge is discharged. Many plants are lagooning and

air-drying sludge and then having it hauled away to be used for soil conditioning.

A two way application of lime, i.e. to a relatively clear water, resulting in a lime precipitate or sludge containing practically no magnesium is now suggested. Lime, in the presence of calcium bi-carbonate and magnesium compounds has a selective action. It reacts first with calcium bi-carbonate and then what is left reacts with the magnesium compounds. Consequently, if just enough lime is added to the water to combine with the calcium bi-carbonate a precipitate or sludge practically free from magnesium and suitable for reuse in water softening can be produced. Two mixing tanks and two settling basins are required to carry out the process.

The procedure suggested is:

1. Add lime at entrance of No. 1 mixing basin in quantity sufficient only to combine with the calcium bi-carbonate content of the water. Soda ash may also be added in sufficient quantity to react with the calcium sulphate present.
2. Stir; settle in settling basin No. 1; reclaim the deposit by burning it into lime.
3. At entrance of second mixing tank, a second dose of lime is added to the water in sufficient quantity to precipitate the magnesium compounds. It is then stirred in mixing tank No. 2; settled in settling basin No. 2; carbonated and filtered. The precipitate magnesium hydroxide, or perhaps a mixture of magnesium hydroxide and calcium carbonate from the second basin may possibly be used for making some magnesium compound of value.

### OVERCOMING LIMITATIONS OF THE LIME-SODA PROCESS

It was shown, as far back as thirty-four years ago at Oberlin, that with six days settling the alkalinity of lime softened water could be reduced to 30 p.p.m. The alkalinity and magnesium content was however not so very high at Oberlin. (Alkalinity 170 p.p.m. and magnesium 25 p.p.m.) When operations started at plants having higher alkalinities and more magnesium to remove than was present in the Oberlin supply and with further handicap of having shorter settling periods, it soon became evident that lime softening reactions do not proceed in an orderly fashion as indicated by mathematical equations, and that alkalinity reductions to 50-60

or 70 p.p.m. were perhaps the best obtainable. Mr. Koyl, Supt. of Water Supply for the Great Northern and Chicago, Milwaukee & St. Paul Railroads, was perhaps the first really to soften water down to the theoretical limit. He reduced the hardness to a new low figure. The process consisted in over treating with lime (usually 2 to 6 grains excess) and then neutralizing the excess lime with soda ash, converting all the causticity to sodium hydroxide. The soda ash required was that necessary to combine with the non-carbonate hardness and the excess lime added. This treatment was satisfactory for producing low hardness water for boiler feed purposes, but was not suitable for municipal use. It was tried in rather a mild fashion for a short time in Columbus, but the consumers complained of the caustic or alkali taste of the water and it was given up.

### SPLIT TREATMENT

Split treatment produces a greater reduction in hardness of a given quantity of raw water with a given quantity of chemicals than does the addition of the same quantity of chemicals to the total supply. Split treatment consists in over treating as large a portion of the hard water with lime as possible to get maximum reduction of carbonate hardness (especially magnesium) and then neutralizing the excess lime or causticity with raw water.

Since the development of recarbonation, split treatment is not now as advantageous because with recarbonation the entire quantity of water can now be over treated with lime and the excess neutralized later on in the process with $CO_2$ gas.

### SUBSTITUTING ZEOLITE FOR SODA-ASH

As an economy measure the substitution of zeolite for soda ash to remove non-carbonate hardness has been used. Where salt can be obtained at a low cost it is usually more economical to remove non-carbonate hardness by means of zeolite than by soda ash. Several municipal lime-zeolite softening plants have been built and are in operation.

### TREATMENT OF FLOOD WATER

We found out, by experience, at Columbus that the usual procedure of softening water cannot be followed successfully in treating extremely muddy water requiring the heavy use of coagulants. During flood periods practically all of the gold fish in homes and aquariums

and minnows in bait stores failed to survive in the treated water. In addition, the water had a bad taste. This happened repeatedly for many years until the proper treatment for flood waters was devised. It was learned that in the treatment of flood waters provisions had to be made to coagulate and remove the bulk of the mud before softening chemicals were added.

The flow line should be: (1) Add coagulant; (2) mix; (3) settle; (4) add softening reagents; (5) mix; (6) settle; (7) carbonate; (8) filter; and (9) adjust pH if necessary.

### RECARBONATION

Although recarbonation for lime softened filtered water, was provided for in the very first municipal plant built in North America, and although Mr. Whipple and others predicted that, if water softening plants were to develop, recarbonation must receive careful attention, yet very little was done about it outside of laboratory experiments until 1921. At that time Nicholas S. Hill, Jr. constructed the first workable and controllable means of applying $CO_2$ to the effluent from water softening settling basins. The purpose of recarbonation, as originally designed, was to convert the carbonates of lime and magnesium and lime hydrate, in the softened water, into the bicarbonates and thus prevent incrustation of the sand and deposits from forming in the distribution system.

A little later on recarbonation plants, similar to the one built by Mr. Hill at Defiance, Ohio, were installed at Newark and Piqua (Ohio) and at other places. After these plants were put in operation it was soon discovered that the addition of carbon dioxide to the softened settled water, when added in sufficient quantity to change the normal carbonates to bicarbonates dissolved the colloidal and basic carbonates and increased the alkalinity or carbonate hardness of the softened water. This introduced a new problem. It seemed to be a step backward to introduce into the process something that hardened rather than softened the water.

Mr. James Montgomery, Chemist in Charge of the Piqua, Ohio plant noticed that when enough excess lime was used to precipitate completely the magnesium and then just enough $CO_2$ was added to neutralize this excess, that the alkalinity of the softened filtered water was reduced to the theoretical solubility limit, that is to about 18 to 22 p.p.m. When this figure is compared to 30 obtained at Oberlin, after 6 days settling, and with the average of 50 to 60 ob-

tained at the Columbus, Ohio plant over a period of about 20 years, it is apparent that this method of treatment constituted a distinct forward step in municipal water softening.

The big drop in alkalinity unfortunately takes place as the water passes through the filters, and the sand grains incrust and become cemented together just as they did before the adoption of the recarbonation process. Operators have been willing, however, to sacrifice the filter sand in order to obtain the low hardness results. At the present time only a few recarbonation plants are operated for the purpose of preventing incrustation of the sand beds. It is believed to be more economical to replace the sand in the filters approximately every 8 to 10 years, than to sacrifice the removal of the hardness.

It is known that, if sufficient time is allowed between the recarbonation basin and the filters, the reactions will be completed and the colloidal precipitates will crystallize and settle out of solution and will not be carried to the filters. So far as we know this is the only workable means known for eliminating filter sand incrustation. At one plant where two or three days is allowed for settling following recarbonation, filtration is not used.

## THE CURRENT PROBLEM

The most pressing problem in water softening today is: How can the colloidal precipitates produced in the recarbonated water be removed quickly and ahead of the filters?

It is known from the results of laboratory experiments that the addition of precipitated calcium carbonate will successfully drag these colloids out of solution, but the addition of lime sludge followed by stirring and settling in plant practice so far has not been very successful. We are endeavoring to solve this problem. Experiments have been made at the Columbus plant with the Spaulding tank. The unit used was very small, probably not properly designed, but it was found to be extremely hard to control the sludge levels. Experiments are now being run with an International Filter Co. accelerator. The removal of these precipitates by blanket filtration through sludge rather than by adsorption on the sand filters is much to be desired.

Further study is required to determine exactly what should be the alkalinity and pH value of a lime softened water so as to make it behave properly in the distribution system and in household plumbing.

I am glad to report that with our present knowledge of water softening, alkalinity and pH value can now be maintained at almost any desired figure. This was not true only a very few years ago. It is the policy at the Columbus, Ohio plant to maintain a total alkalinity of about 35 p.p.m. and a phenolphthalein alkalinity of about 9 or 10 p.p.m. in the softened filtered water. This produces a pH value approximately 9 and is just about sufficient to make certain that the water is in chemical balance to calcium carbonate.

# CONDITIONING OF WATER SOFTENING PRECIPITATES*

By Charles H. Spaulding

*(General Superintendent)*

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A friend who has been studying musical harmony recently remarked that he believed musical compositions were written first and the rules of harmony made to fit. This observation is close to the truth in fields other than music. Perhaps most rules and theory are but rationalizations of things already done. Doubtless there are scientific principles involved in the conditioning of water softening precipitates which are brought to light by the process and apparatus I am about to discuss. On the other hand, you are more interested, I hope, in hearing as simply as possible the nature of the process, how and why it was developed and what it has accomplished.

## NEW APPLICATION OF OLD PRINCIPLES

The principles involved in the Precipitator, a name which seems to best describe the apparatus, have been known for a long time and the only wonder is that they were not applied sooner in the water softening process. Principle number one is that to produce a saturated solution under a given set of conditions, the solution phase and the solid phase must be brought into intimate contact under those conditions. In the laboratory, this ordinarily implies agitation of the liquid with a large excess of the solid in finely divided form in order to bring about the desired saturation equilibrium. This technique is too well known to be discussed. It is enough to point out that the completion of softening reactions with lime or lime soda is essentially the production of a saturated solution for a given set of conditions. The second principle is also one of laboratory technique. It is common practice to stir precipitates before filtration. By this means we find that the precipitates become coarser. Small particles agglomerate into larger coagulated masses. Filtration is conse-

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quently more rapid and efficient. It is true that this principle has gained considerable recognition during the past 15 years. The Precipitator represents an effort to apply both of these principles more effectively.

### SLUDGE RETURN

It may be said at the start that the practice of returning sludge was not the approach. From the experience of the writer with returned sludge, he was under the impression that it was not usable in a water softening plant handling surface water, whatever its adaptability might be in some special cases. My experience has been that sludge which has settled to the bottom of a basin, when scraped together and returned to the raw water, imparts disagreeable odors and tastes, due in part to fermentation within the sludge; furthermore it seems to impart a fine turbidity difficult to remove in the usual coagulation process. Hence the return of settled sludge did not seem to be a promising avenue to investigate when the Precipitator was conceived. On the other hand, I was particularly interested in the possibility of stabilizing water which had been treated with excess lime and recarbonated. As is well known, such water is extremely unstable in that the unprecipitated calcium carbonate separates readily and rapidly upon filter sand or pipes and fittings through which the water flows and yet it does not separate rapidly in a quiescent basin nor even if agitated, unless a considerable quantity of solid particles is added to give the necessary condition for reducing the dissolved calcium carbonate to the point of saturation. It had been observed that the stabilizing effect was easily obtained by downward filtration through filter sand coated with calcium carbonate. It seemed likely that the same effect could be obtained by upward filtration through suitable material, which might be either calcium carbonate of desirable size or light material coated with calcium carbonate.

### PRINCIPLES OF DESIGN

The use of a conventional filter in upward filtration could not be considered. Any sort of an underdrain system would be fouled with scale very quickly in such service. The first concept therefore, in the evolution of the Precipitator took the form of a funnel through which the water to be stabilized was forced backward and upward. It was visualized that for any particle size, upward velocities might be found in a horizontal plane through the funnel sufficient to sup-

port the particles without carrying them higher or out of the funnel. The expanding cross-sectional area of the funnel would permit such particles to find an equilibrium automatically at various rates of flow thus providing a degree of flexibility. By this means it was hoped to provide the necessary contact of solid and liquid without the disadvantages attendant upon filtering unstable water and with more complete results.

A simple funnel however would not be sufficient. One can easily imagine the difficulties that would arise due to temporary interruptions of the flow through such an arrangement. In order to make it workable it would be necessary to maintain the stabilizing material in suspension in the liquid regardless of the rate of upward flow. Such a scheme therefore necessitates an agitated zone below the funnel in which the solids are maintained in suspension ready for transfer by the flow of the liquid into the funnel. The apparatus first put into operation in the laboratory consisted of a funnel with stem removed, (the settling compartment) supported in a cylindrical jar, (the mixing compartment) having the necessary agitator mounted on a shaft passing through the funnel. Treated water entered the mixing compartment and the clarified stabilized water was siphoned from the surface of the funnel. Various materials, including powdered coal and activated carbon were employed as stabilizing materials. The scheme worked in the laboratory very encouragingly, particularly from an hydraulic standpoint which was the point of most concern. The suspended material assumed a position in the funnel resembling filter sand during a backwash and rose or fell with increases or decreases of rate of flow through the apparatus. From a chemical standpoint also, its effectiveness was at once proven. Moreover, the objections which had prevented the use of returned sludge did not apply to *retained* sludge which had not previously been allowed to settle. In other words we could obtain from such an apparatus as clear and odorless an effluent as from a conventional coagulation tank. Another gratifying thing was that the rate of precipitation and coagulation were so increased that in a laboratory model we could accomplish results in a fraction of an hour which were taking eight to ten hours in our plant.

## PLANT TESTS

After operating this device in the laboratory for several weeks and observing its behaviour, the next step was to install equipment on a plant scale. This was done as a preliminary to the design of the new

softening plant for which studies of basin design had previously been started. The Precipitator was built into a circular mixing chamber, one of two which were operated in parallel. Thereafter the behaviour of the Precipitator could be compared directly with the conventional system, which included mixing and settling in two separate operations, the sludge being removed continuously from a clarifier on one side and on the other as a thin slurry from the Precipitator. The comparative results have already been mentioned in a paper on "The New Springfield Water Purification Plant" (1). It was shown that the new combination tank which I am calling the Precipitator accomplished better results in an hour than the old system was accomplishing in eight to ten hours. It is true that periods of settling have been shortened in recent practice, but mixing or coagulation time has been increased generally while the results have not been comparable with those being described.

It may be pointed out here that the Precipitator which is now being referred to does not consist of a funnel superimposed upon a mixing tank. The original idea has been modified in order to secure more economical design in units of large capacity as will appear. The capacity of the Precipitator is dependent upon the area. In other words, we must limit ourselves to certain reasonable vertical velocities.

### SIZE AND CAPACITY FORMULAE

Since the velocities decrease as the water rises, we must adopt a particular horizontal plane at which we define the velocity. It is convenient to adopt the surface area for this purpose, although it is apparent that there is actually no vertical flow at the surface. However, just below this plane the upward flow may be assumed to have reached its minimum. We have adopted the limiting rate of 2 inches vertical rise per minute solely as a result of experience. As a matter of fact this rate has been exceeded more than 100 percent under certain conditions. On the other hand, certain precipitates such as may be obtained with coagulant alone will not permit a vertical rise as high as 2 inches per minute. This is a matter which can only be determined empirically.

It has been stated that the rated capacity of the Precipitator is based upon the area. Algebraically,

$$C = 240\,A$$

where $C$ is the rated capacity of the tank in cubic feet per day and $A$ is the area at the surface in square feet. For a circular tank,

$$A = \pi r^2$$

and the required radius for a given rated capacity is therefore expressed by the formula

$$r = \sqrt{\frac{C}{240\pi}}$$

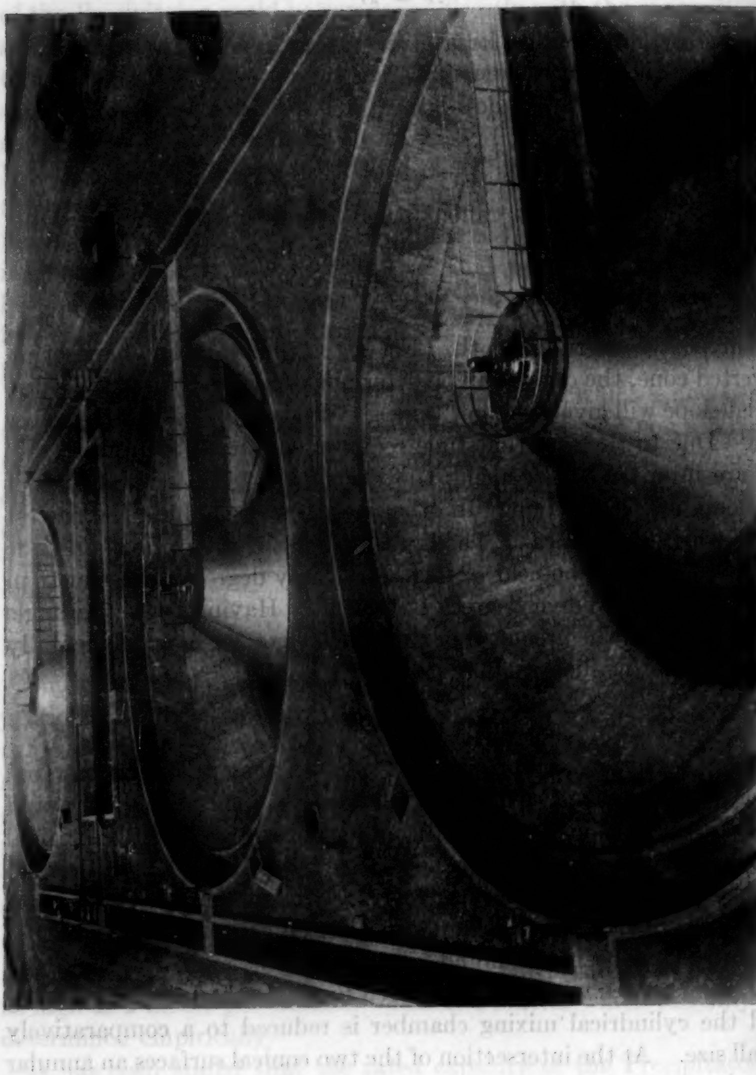
Converting $C$ to million gallons per day and substituting the value of $\pi$ we find that

$$r = \sqrt{177C}$$

Cur next concern is depth. For a tank having the shape of an inverted cone, the depth will be determined by the slope of the sides. A flat slope will give a shallow tank. A steep slope will give a deeper one. Due to the fact that time requirements have been decreased, we are interested in reducing the size of the tank for the sake of economy of construction. In other words, we would use the flattest slope which is practical. The limiting angle of the slope of the side has been found to be near 52 degrees. Sixty degrees is steeper than necessary. Forty-five degrees is too flat. Having fixed the angle of the side and the area of the base of the cone, we have incidentally fixed its depth which we may call $h$.

$$h = r \tan 52° = 1.28\, r$$

Having previously established the value of $r$ on the basis of rated capacity of the plant we find that the depth of the Precipitator would become excessive in sizes above 1 million gallons rating if the original simple form of funnel in cylinder were used. Hence the original design was modified to secure the same capacity in the larger sizes with approximately half the depth. In effect, the bottom half of the funnel has been cut off and turned upside down within the upper half. The space within this inverted funnel is then used for mixing and the cylindrical mixing chamber is reduced to a comparatively small size. At the intersection of the two conical surfaces an annular port is provided for passage of the water from the mixing compartment. This port is cut into segments by radial baffles in order to prevent transfer of circular motion to the settling compartment.



PRECIPITATOR DURING CONSTRUCTION, SPRINGFIELD, ILLINOIS

## FACTORS AFFECTING RATE OF FLOW

The question arises—how does this system differ from any other upward flow tank? Coagulation and settling tanks involving upward flow principle are not novel. The novelty consists in supporting the precipitate in the settling compartment by means of vertical velocities which are maintained relatively even at all points in a horizontal plane. This is an essential feature, because any tendency to unbalance one side of the tank as compared to another would result in greater velocities in certain spots with turbid and unstable water emerging from the tank. The type of agitation therefore is such as to overcome or prevent unbalancing of upward pressures. The problem is similar to that involved in the upward flow of water in filter washing but instead of using an underdrain system to equalize the vertical flow, the balance is maintained by kinetic energy derived from the agitator. As a matter of fact, this principle is vital to the success of the Precipitator.

Returning for a moment to permissible rating of capacity, this rating, as has been pointed out, is an arbitrary figure, based on experience under given conditions. There are a number of factors which determine permissible vertical rise, but none of these are definitely evaluated as yet because they have not been isolated in plant practice. We know that a precipitate of one coagulant will permit a higher upward velocity than another. A precipitate consisting entirely of calcium carbonate such as is formed by recarbonating water which has been treated with excess lime and clarified will be crystalline in character and will build up particles of which the size is only limited by the ability of the agitator to maintain the precipitate in suspension. The allowable upward velocity in this case is much higher than 2 inches per minute. It may be 4 to 6 inches. On the other hand, magnesium hydroxide forms a lighter precipitate while a precipitate consisting chiefly of aluminum hydroxide apparently floats more readily than either of the others.

Again temperature is an important factor and for temperatures in the neighborhood of freezing (which are the most difficult), we have found that our rating is approximately correct. On the other hand, at high temperatures, the upward velocity may be doubled without any embarrassment and even with improved results because without such increased velocities the sludge would require more violent agitation in order to prevent settling out. At temperatures above 15°C. therefore we can put our entire output through two tanks in

series. For a considerable part of the day our pumping rate is 12 m.g. The detention period in each of the two Precipitators at this rate is 45 minutes, the upward velocity 4 inches per minute. At present, we are using sugar sulphate of iron (copperas) as coagulant. We have used sulphate of alumina and we have also used excess lime treatment in lieu of coagulant. In brief laboratory tests we have used silica (alone and in conjunction with other coagulants) and we have also tried Ferrisul, a product of the Monsanto Chemical Co. The value of these chemicals is roughly independent of the type of mixing and clarifying in so far as we have observed. Whatever advantages one possesses over the other has appeared in ordinary jar tests as well as in the Precipitator. We conclude therefore that while the Precipitator brings out the characteristics of the coagulant, it does not show any favoritism. On the other hand, it is obvious that whereas a long period of mixing or clarifying may conceal inherent advantages of one or another coagulant, these advantages stand out sharply when the total time of mixing and clarifying is reduced to 30 minutes.

## SLUDGE CONCENTRATION

The behaviour of the sludge in the sludge concentrator which is an adjunct of the process may be of interest. The concentrator has one influent from each Precipitator, taken from the bottom of the Precipitator and discharging by gravity into the concentrator in such a way as to create little or no disturbance. The concentrator has two effluent lines; one for clear water through a pump of 100 gallons per minute capacity which discharges the supernatant liquid into the raw water stream at the dosing well; the other for the concentrated sludge which is discharged upon the sludge bed and is the ultimate disposal. The sludge pump is a Barnes pump of 75 gallons per minute capacity which we are operating at the present time at 37 gallons per minute for a period of 8 hours each day. The clear water pump is allowed to operate sixteen to twenty-four hours daily. Flow of sludge from the Precipitators takes place only when one or more pumps are in operation. Concentration of the thickened sludge which has passed through the concentrator has varied from 5 to 15 percent. The average for the month of May has been 10.3 percent. Concentration of unthickened sludge normally varies from 1 to 2 percent. The detention time of the thickener, based on the present 24 hour cycle of operation is 8 hours. Under low temperature

conditions this may be reduced 50 percent or more by the necessity of handling thinner sludge. The plant averages up to now are of no great significance because we are still getting acquainted with the new procedure and investigating various possibilities. However, a brief summary (table 1) of operating results for the first 6 months may be interesting.

The time of detention in the mixing compartment of the Precipitator is 30 minutes at the rated capacity of each unit (6 million gallons). The nominal detention time in the settling compartment is 60 minutes. The results cited include operation up to 200 percent of the nominal rating.

TABLE 1

| | LIME | COAGULANT | ALKALINITY (TEMP. HARDNESS) | | | TURBIDITY | | |
|---|---|---|---|---|---|---|---|---|
| | | | Raw | Applied | Filtered | Raw | Applied | Filtered |
| | *p.p.m.* | *p.p.m.* | | | | | | |
| December | 123 | 9 | 132 | 33–8 | 29–6 | 9.6 | 8 | 0.05 |
| January | 89 | 16 | 125 | 39–17 | 32–13 | 15 | 7 | 0.20 |
| February | 73 | 17 | 111 | 40–21 | 36–19 | 40 | 3.5 | 0.05 |
| March | 63 | 18 | 104 | 40–20 | 35–18 | 46 | 4.3 | 0.003 |
| April | 72 | 18 | 102 | 38–19 | 33–17 | 44 | 2.6 | 0.02 |
| May | 70 | 18 | 104 | 37–18 | 32–16 | 36 | 2.3 | 0.01 |

## EFFECT UPON RECARBONATION

So far, I have discussed principally the effect of the Precipitator on the time required for precipitation and coagulation. This is important because reduction of time reduces the necessary size of basins. Another advantage is the stabilizing effect of the Precipitator which is so marked that recarbonation becomes unnecessary. Possibly only those most familiar with lime softening reactions realize that the introduction of recarbonation as a step in water softening was definitely due to incompleteness of the softening reactions in the water applied to the filters and even in the filter effluent. Light on this subject was available in a report of A. Elliott Kimberly (2) published in 1908. The operators of the Oberlin plant knew that carbon dioxide could be used to stop softening reactions which were incomplete and they also knew that excess lime could be used to help drive the reactions to completion.

The life of recarbonation is approaching a complete cycle in four

steps. First, incomplete softening; second, excess lime treatment; third, recarbonation to correct 1 and 2; fourth, end of recarbonation due to complete reactions. However, we cannot ignore the fact that magnesium removal requires caustic alkalinity and it may be that recarbonation will still be required for high magnesium waters especially if soda ash is used for non-carbonate softening.

During recent months we have not used recarbonation and it remains to be seen whether excess lime treatment will ever be required or even desirable at this plant. If it were desirable to do so it would be comparatively easy to reduce the temporary hardness of the softened water to 10 or 15 p.p.m. and to have a stable water flowing to the filters. At present we are sure that this would be fatal from the standpoint of red water. It is necessary that the filtered water carry a total alkalinity in the neighborhood of 30 p.p.m. and a pH high enough to prevent re-solution of calcium carbonate. This brings us into the pH zone between 9.6 and 10.

## RAPID FILTER RATES

A third feature of the performance of the Precipitators which has been gratifying is the rapid rate of filtration now feasible, and the consequent reduction of filter area. It should not be surprising to an analytical chemist to be told that by proper preparation of the precipitate, a much coarser filter and a much higher rate of filtration can be used with equal results. This is everyday observation in routine laboratory precipitation. To others, however, not so familiar with laboratory technique, doubtless it is a little startling to have the filtration rate stepped up 100 percent with sand sizes of 1 mm. or more instead of 0.4 to 0.6 mm. A well known engineer (and one whom I greatly respect) raised the question as to how we were able to anticipate with certainty that a double rate of filtration through such coarse sand as we are using would prove satisfactory. Apparently we had taken chances. The answer to the question is that we did not jump to this conclusion but crept to it by experience of 10 years at the Riverside Plant. There it was that the filter sand had been allowed to increase in size, year by year, from its original 0.4 mm. to more than 1 mm. size. Year by year we observed the softening value of the filter sand and the fact that the filters were continuing to function equally well as before. It was not possible to force the water through the old filters at quite the rate which is now employed at the new plant, but it was possible to approach suffi-

ciently near this rate to show that 4 gallons per square foot per minute is entirely feasible, when a lime softened water is thoroughly prepared for filtration. Now we find, as a matter of fact, that for many purposes such treated water can be made clear enough in the Precipitator to dispense with filters entirely. We have indeed delivered water to the filters with turbidities measured in tenths of 1 p.p.m., and this without any special effort.

In conclusion, the Precipitator is presented as a rational and practical device for the conditioning of water softening precipitates to the end that we may use most effectively the simple chemical reactions of water softening.

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# HOLLYWOOD'S ZEOLITE WATER SOFTENING PLANT*

By W. Austin Smith

*(Consulting Engineer, Jacksonville, Fla.)*

The Hollywood water softening plant is of particular interest as it is the first municipal water softening plant of the zeolite type to be installed in Florida. This plant has been in continuous operation since last August and the results that have been obtained are excellent.

The water supply is drawn from three wells of an average depth of 120 feet, and analyses have shown that the three waters are practically identical in composition. This water has an average hardness of approximately 17 grains per gallon due mostly to calcium bicarbonate, there being only about two grains of magnesium hardness present and about the same amount of non-carbonate hardness.

The iron content is low, about 0.3 p.p.m., expressed as iron, but the water from the wells does contain some hydrogen sulphide and aeration is, therefore necessary. The aerator is mounted on the top of a storage tank of 100,000 gallons capacity. Formerly the well pump elevated the well water to the aerator, on top of the storage tank. The water dropped from this aerator into the storage tank and then flowed, by gravity, to the pump house. There it was chlorinated and then pumped to the city mains by means of high service pumps.

While this water was entirely satisfactory for drinking purposes, its hardness made it undesirable for other purposes and it was decided to soften it to approximately 5 grains per gallon. In selecting the method of softening and the type of equipment to be employed, a close study was made of the relative costs and applicability of the lime alum process and the zeolite process.

This study showed that the lime-alum process, due to its higher initial cost, its sludge disposal problem and its higher total costs for attendance and operation, was not so applicable as the zeolite

* Presented at the 1937 meeting of the Florida Section.

process. With the zeolite process, the initial cost was lower; the sludge disposal problem was eliminated; the total cost for attendance and operation was lower and it could be installed very simply by placing the zeolite water softeners between the elevated storage tank and the service pumps.

Furthermore, it was decided that a completely automatic zeolite water softening plant would be much more desirable than a manually operated plant. For the automatic plant would eliminate the possibility of incorrect operation, due to the human factor; would minimize the amount of attendance required; would assure the highest degree of utilization of the salt required for regeneration and would prevent the possibility of undue amounts of water being used for backwashing and rinsing operations.

Specifications for this type of plant were accordingly drawn up and, after approval by the P.W.A. authorities was secured, bids were called for. The successful bidder was the M. and M. Construction Company of Miami, and the softening equipment was furnished by the Permutit Company of New York.

The plant consists of one 11-foot diameter x 6-foot deep steel coke aerator; three 9-foot diameter x 10-foot high Permutit vertical type automatic zeolite water softeners; one 48-inch diameter x 48-inch brine measuring tank; one concrete wet salt storage basin; and is equipped with automatic by-pass valves and rate of flow indicators. The type of zeolite employed is "Super-zeo-dur", which is a high capacity, green-sand base zeolite.

The equipment is housed in a new building 30 feet long by 20 feet wide by 22 feet high. This building was erected on ground located between the storage tank and the pump house.

This plant will soften 1,500,000 gallons per day of water having a hardness of 17 grains per gallon, producing a mixed effluent having a hardness of 5 grains per gallon. Room was left in this building to accommodate, whenever required in the future, a fourth water softening unit, which would increase the capacity to 2,000,000 gallons per day.

In softening the Hollywood water, twelve parts of the completely softened water are mixed with five parts of the hard water, thus yielding seventeen parts of 5-grain water. Also, if at some future date, it might be desirable to produce a somewhat softer water, this could easily be accomplished by softening a relatively larger proportion of the water and mixing this with a relatively smaller proportion

of the hard water. In this way, zeolite water softening plants present a high degree of flexibility. At Hollywood, this proportioning of the water is accomplished by means of automatic by-pass valves.

The automatic operation of the water softening units is accomplished by means of a motor driven multiport valve, the rotation of which to the various settings is accomplished by means of electrical controls. Briefly, the volume of water softened on each softening run is measured and governed by means of a meter, equipped with an electrical contact head.

At the end of the softening run, the meter makes an electrical contact, establishing a current which actuates a small motor mounted on the valve. The motor rotates the valve to the backwash position, at which point the contact is automatically broken and the motor stops.

The length of the backwashing period is governed by an adjustable time-switch, and the rate of flow of the backwash water is governed by a butterfly valve operated by the level of the water behind a fixed orifice plate in a sump.

At the end of the backwash period, the motor is again automatically actuated so as to rotate the valve to the brine position. Then, by means of a hydraulic ejector, a pre-determined amount of a saturated solution of common salt is injected into the softening unit. The salt acts on the zeolite causing it to give up its calcium and magnesium content, in the form of the very soluble chlorides, which may be discharged into any convenient drain.

Simultaneously, the equivalent amount of sodium is taken up by the zeolite, thus restoring it to its original condition. The amount of salt solution is governed by a float switch. When the correct amount of salt has been injected into the softener, this float switch makes a contact, thus actuating the motor so that it rotates the valve to the rinse position.

In the rinse position, an electric time switch governs the rinse period, while a second butterfly valve regulates the rate of flow of the rinse water. The rinse serves to remove the excess salt and soluble by-products to the drain. At the end of the rinse period, the motor is again actuated so that it rotates the valve to the softening position, thus returning the softener unit to service. In the meantime, the meter has automatically been reset for the succeeding softening run.

In this way, all operations are automatically accomplished,

both on the softening run and on the regeneration, which last term covers the backwashing, salting, and rinsing operations. The salt solution is formed in a large concrete wet salt storage basin, which has a capacity of two carloads of salt. From this, the saturated salt solution is automatically pumped into the brine measuring tank as required.

About the only duties that devolve on the operator of an automatic zeolite water softening plant are to see that the supply of salt in the wet salt storage basin is renewed when required. The operation of the Hollywood plant has been very satisfactory and the citizens are very well pleased with the quality of the softened water which it furnishes.

# SARASOTA'S AUTOMATIC SEA WATER REGENERATION ZEOLITE WATER SOFTENING PLANT*

By Charles E. Richheimer
*(G. A. Youngberg & Associates, Inc.)*
*(Consulting Engineers, Jacksonville, Florida)*

## INTRODUCTION

This paper will present the various problems which confronted the City of Sarasota in its endeavor to obtain an adequate and dependable supply of potable soft water for the municipal water works system, as well as the problems which confronted the engineers in designing and constructing the automatic sea-water regeneration zeolite water-softening plant.

Sarasota is now installing an automatic, sea-water regeneration zeolite water softening plant. This plant is, to the writer's knowledge, the first municipal plant of its type.

The operating costs of this softening plant, per unit of hardness removed, will be only a fraction of the operating costs of any other type of municipal water softening plant. As an illustration, a lime soda water softener, in this location, would have had an operating cost of $193.00 per million gallons for chemicals alone.

With sea-water regeneration, Sarasota's municipal zeolite water softening plant will have an operating cost of only $14.00 per million gallons. This includes the costs of pumping, coagulating, filtering and chlorinating the sea-water required and also the costs of pumping the water through the zeolite softeners and maintenance of the zeolite units at their full capacity.

Possibly, with some bicarbonate waters, which require only lime for softening, the comparative operating costs would not be quite so marked but, in any event, they would be only a fraction of the operating costs with any other method of water softening.

## RAW WATER INVESTIGATION

Before deciding on the method of treating this water, the engineers made an exhaustive study of various raw water supplies and the

* Presented at the 1937 meeting of the Florida section.

possibilities of obtaining and treating them. These sources of supply were:—

(A) Shallow wells;

(B) Surface waters impounded;

(C) Wells in the Hawthorn Formation, at depth of from 300 to 400 feet, the water from which would have a total hardness of from 16 to 35 grains per gallon;

(D) Wells in the Tampa and Ocala Formations, at depth from 450 to 750 feet, the waters from which have a total hardness of from 50 to 70 grains per gallon.

### CONCLUSION AS TO SOURCE OF RAW WATER

Wells 400 feet deep, cased for the first 200 feet, were recommended as a source of raw water. The softening plant was designed to treat waters from the present wells (of 65 to 70 grains per gallon hardness) and also designed so that any decrease in hardness of the raw water will merely mean cheaper operating costs, coupled with proportionally increased plant capacity.

*Shallow Well Water:* Tests were made on over forty-one (41) possible sources of raw water from shallow wells and it was found that these wells, covering fairly large rural sections in each case, produced water having an average total hardness of ten to eleven grains per gallon; too hard to be satisfactory without treatment.

It might be possible to pipe this water to the plant (at an operating cost of approximately $3,000.00 per year for electric power for the pumps alone) and its adequacy could be insured by using the present objectionably hard deep well water (approximately 62 grains per gallon) as a standby source. However, no softening equipment could be built under the project as the total amount of money allotted by PWA was not sufficient.

The fluoride content of a shallow well (27 feet deep) that produced a soft water was found to be 0.9 parts per million and the character of the superficial material overlying the Hawthorn formation indicates that phosphates are present *above* the Hawthorn formation, as well as probably in the upper levels of the formation itself. Another shallow well fifteen feet deep was tested, and this fifteen foot well showed a water containing 0.45 parts per million fluoride.

*Summarizing:* The water obtained from shallow wells was too hard to be satisfactory; no softening equipment could be installed under the PWA funds allotted; operating costs of pumps alone would be

$3,000.00 per year (with relatively high service and maintenance costs to be added); and the decrease in the fluoride content of this water, over water obtained from wells 400 to 450 feet deep, was negligible.

*Surface Waters (Impounded):* Five possible sources of impounded water were investigated. The lowest estimated cost was $138,000.00 for construction work alone, not including any pumping costs necessary to pump the impounded water for a distance of sixteen miles and also not including any secondary treatment which would probably be required for this water.

*Wells in the Hawthorn Formation:* These wells, 300 to 400 feet deep yield, in good quantities, water that is from 16 grains to 33 grains per gallon total hardness and the fluoride content of one of these wells 450 feet deep, in constant use, was only 1.1 part per million.

These wells would be free from any possibility of saline infiltration for generations to come as the present ground water level at the municipal plant is approximately at Elev. +25.0 which results in the impossibility of saline infiltration below Elev. −1,000.0.

*Wells in the Tampa and Ocala Formation:* These wells, from 450 feet to 750 feet deep, yield water in good quantities, but the additional yield obtained by going deeper than 400 feet with a well in Sarasota is slight, while the hardness of the water is increased over 100 percent. The three municipal wells are good examples of this. These wells run from 65 to 70 grains per gallon total hardness and their flow is no greater than that from a nearby 450 foot well. The fluoride content of the municipal wells was found to be 1.5 parts per million, which, while not too high, indicates that the fluoride content of rock wells in Sarasota may increase slightly with depth.

The draw-down in the municipal wells was found to be much too great for efficient operation of horizontal centrifugal pumps.

### MODIFICATIONS TO PRESENT WELLS

On each of three 675 foot wells used at present, a 350 g.p.m. electric vertical type pump, with impeller 50 feet down in the well is to be installed replacing the present pumps which are unsuitable due to the drawdown in the present wells. The present wells are cased to 80 feet; and this casing should be extended an additional 120 feet.

## SOFTENING PLANT INVESTIGATION

It was recommended that a water softening plant, to soften one million gallons of water per day down to an effluent of six grains per gallon hardness be installed; the plant to be designed to have this capacity where the hardness of raw water is 65 to 70 grains. Any decrease in hardness will result in decreased operating costs. No changes in the plant itself will be necessary. This plant was constructed under a guaranteed performance bond.

The raw water was to be taken from the deep wells; aerated, and allowed to stand in one of the present reservoirs; then passed through the softening plant at a steady rate into the second reservoir from which it would be pumped into the mains with the elevated tank "floating" on the line.

The softening plant was to be a zeolite exchange type of plant using sea water for regeneration. This type of plant was chosen primarily because it is the only softening treatment that would be economically possible with the present raw water. However, there are other considerations which are listed below:

a) A zeolite plant can be made *fully* automatic, requiring no attendant.

b) Zeolite softening functions on water supplies that vary greatly in composition.

c) Zeolite changes the bicarbonate of calcium and magnesium into sodium bicarbonates, no carbonates or hydrates are present in the zeolite effluent. The water is therefore neutral and no after-precipitates are possible.

d) There is no sludge disposal problem with the zeolite process.

e) The sulphate or chloride hardness of the water, as treated by soda ash, introduces a corresponding sodium chloride content and as the proposed raw water is high in sulphates and chloride there is not any advantage to be gained by the soda ash treatment in this particular case.

f) Constant attention by trained men is not necessary in the zeolite process as there are no chemical feeds to change.

g) Industrial plants using the small industrial type zeolite process water softeners in Sarasota at the present time report complete satisfaction with them.

## SOFTENING METHODS

In deciding on the method of water softening to be employed, both the lime-soda process and the zeolite process were studied. As the well supply, now in use, has a bicarbonate hardness of only 7 grains per gallon and a non-carbonate hardness of about 58 grains per gallon, it was evident that no reduction in the total solids would be effected by the lime soda process. Therefore, the effluents from either the lime soda process or the zeolite process would be practically identical in composition.

The lime-soda process, however, would have a very high operating cost, $193.00 per day, and would produce, as a useless by-product, some 38 tons of wet sludge per day, which would present a very serious sludge disposal problem.

The zeolite process, with sea-water regeneration, would have an operating cost of only $14.00 per day. In addition, no sludge is produced by the zeolite process but only soluble salts, instead, which can easily be flushed down any drain, no sludge disposal problem being involved.

## DETAILS OF ABOVE ESTIMATES

*Cost per million gallons for operation of lime-soda plant*

| | |
|---|---|
| (1) 5,100 pounds of lime hydrate at $12.00 per ton | $35.00 |
| (2) 8,710 pounds of soda ash at $35.00 per ton | 153.00 |
| (3) $CO_2$ gas—50,000 cu. ft. | 5.00 |
| Total for one million gallons | $193.00 |

Note: No labor charges included in this estimate.

*Cost per million gallons for operation of zeolite plant*

| | |
|---|---|
| (1) 400,000 gal. against 100 ft. head—pumping sea water | $4.50 |
| (2) Coagulant-alum (for sea water) | 2.50 |
| (3) Chlorination (sea water and soft water effluent) | 1.50 |
| (4) Pumping raw water through softeners (700 g.p.m. against 30 ft. head) | 3.50 |
| (5) Zeolite maintenance (contract) | 2.00 |
| Total operating cost for million gallons | $14.00 |

Note: Pumping costs figured on basis of 2¢ per K.W. hour for electrical energy.

## PLANT DESIGN

It was, therefore, decided to use the zeolite process and to design the plant so that it would soften 1,000,000 gallons per day of the 65

grain water and could be employed also to soften the water from the Hawthorn formation. As the latter has a much lower hardness, 16 to 35 grains per gallon, naturally, a larger amount of water could be softened with the same equipment. In fact, when operating on the Hawthorn formation water, the daily capacity will be 2,000,000 gallons; and the operating costs will be approximately $10.00 per million gallons.

It was decided that a 6 grain per gallon effluent would be satisfactory. As a zeolite plant is very flexible in operation, an even softer water can be obtained if, at any time in the future, it should be desired.

## SOFTENING PLANT

Specifications covering this type of plant were written, the approval of the PWA authorities was obtained and bids were taken. The successful bidder on the water softening plant was the Ivy H. Smith Co. of Jacksonville, Florida, and the water softening equipment was supplied by the Permutit Company of New York.

The plant consists of four 10 foot diameter by 16 foot high vertical Permutit Automatic Zeolite Water Softeners; four 7 foot diameter vertical Permutit Automatic Sea-Water Filters; one Alum Feed; two Wallace & Tiernan Chlorinators and the necessary control panels, by-pass valves, etc. In designing the building, it was decided to house only the valves and controls and to have the softener and filter shells in the open, on two sides of this building.

Both the softener and filter units are equipped with hydraulically operated valves and the operation of the units is controlled by means of automatic pilot valves, connected to the hydraulically operated valves. The pilot valves are operated by electrical controls so that all operations of both the water softeners and sea-water filters are completely automatic.

The flow of the water is from the wells to an aerator, from which it drops into a raw water storage basin. Pumps pick it up from this basin and send it through the water softeners; the softened water than flows into a soft water storage basin or clear well, from which it is pumped to service by the service pumps.

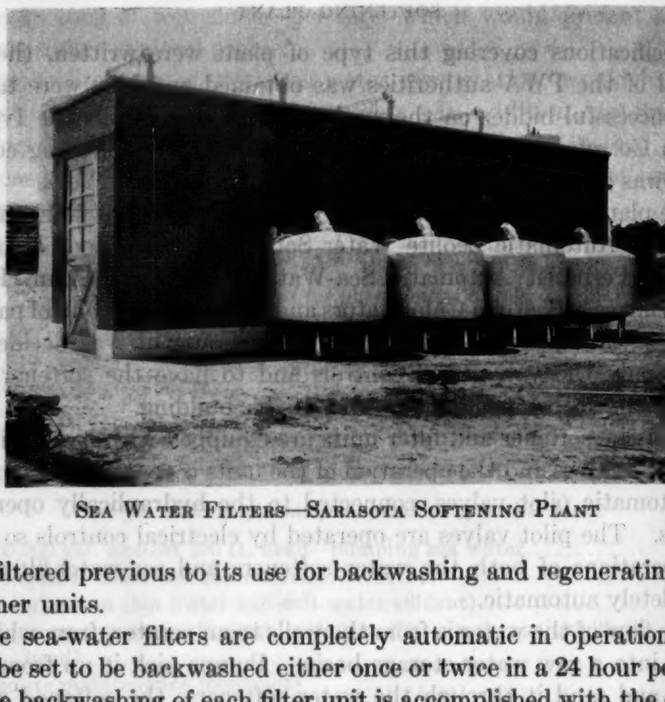
The volume of water softened on the softening run of a unit is measured and regulated by a meter, electrically connected to the pilot valve. At the end of the softening run, the pilot valve automatically cuts the softener unit out of service, regenerates it and returns it to service.

### THE SOFTENING PROCESS

In the usual type of zeolite water softener, there are three steps to this regeneration: (1) backwashing, (2) salting, and (3) rinsing.

In the Sarasota plant, operations (1) and (2) are combined as the softener units are backwashed and salted simultaneously by a strong, upward flow of sea-water. The rinsing is carried out, in the usual manner, by a fairly slow, downward flow of the fresh well water.

The sea-water is obtained from an intake, approximately $\frac{3}{8}$ths of a mile from the plant, and is pumped to the plant through an 8-inch Transite pipe. The sea-water is chlorinated, coagulated with alum

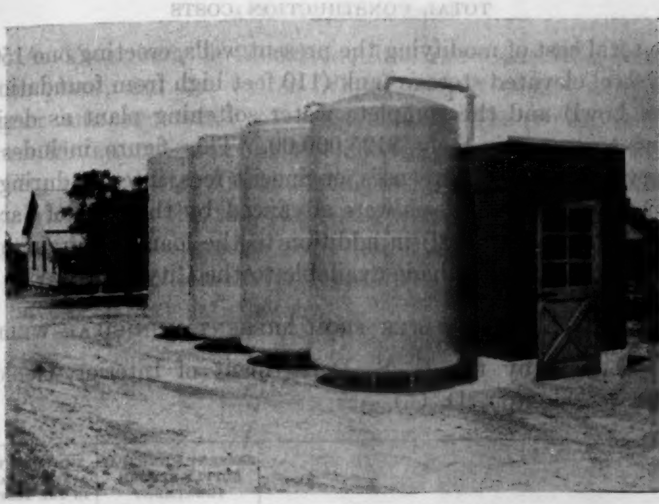


SEA WATER FILTERS—SARASOTA SOFTENING PLANT

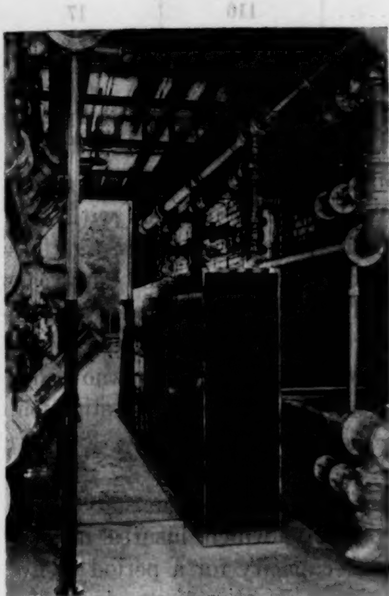
and filtered previous to its use for backwashing and regenerating the softener units.

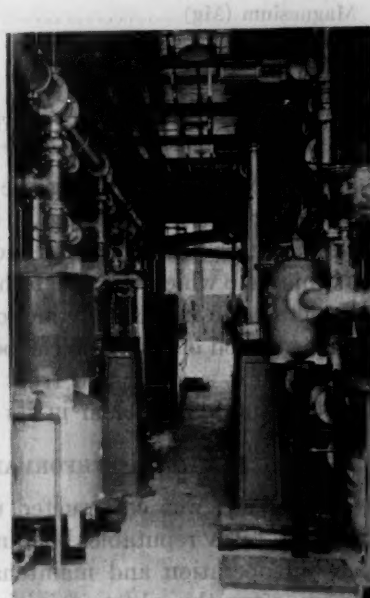
The sea-water filters are completely automatic in operation and may be set to be backwashed either once or twice in a 24 hour period.

The backwashing of each filter unit is accomplished with the clear, filtered, chlorinated effluent obtained from the three other filter units. The backwashing period is, furthermore, regulated by the electrical controls so that it occurs during the rinsing period of regeneration of one of the softener units, when there is no demand for sea-water for the softener units. The period of backwashing on these automatic sea-water filters is controlled by means of a time-switch.



SARASOTA SOFTENER UNITS—CONTROL HOUSE IN REAR





CONTROL HOUSE INTERIOR—SARASOTA SOFTENING PLANT SHOWING CONTROL PANELS, CHLORINATORS AND COAGULANT FEEDERS

### TOTAL CONSTRUCTION COSTS

The total cost of modifying the present wells, erecting one 150,000 gallon steel elevated storage tank (110 feet high from foundations to base of bowl) and the complete water softening plant as designed amount to approximately $125,000.00. This figure includes preliminary reports, legal expenses, engineer's fees, interest during construction and such funds as were advanced by the City of Sarasota (approximately $7,000.00) in addition to the loan and grant in the amount of $118,200.00 made available to the City by PWA.

### ANALYSIS OF RAW WATER FROM PRESENT MUNICIPAL WELLS

(Tests made by the U. S. Department of Interior, Geological Survey, Washington, D. C.)

| | PRESENT MUNICIPAL SUPPLY BEFORE TREATMENT | THE SAME WATER AFTER TREATMENT IN THE TEST PLANT |
|---|---|---|
| | *p.p.m.* | *p.p.m.* |
| Calcium (Ca) | 260 | 42 |
| Magnesium (Mg) | 116 | 17 |
| Bicarbonate ($HCO_3$) | 163 | 175 |
| Sulphate ($SO_4$) | 852 | 832 |
| Chloride (Cl) | 168 | 175 |
| Fluoride (F) | 1.6 | 1.0 |

The above tests showing the reduction of fluorides by the zeolite softening process, were again checked by the engineers with substantially the same results and we are carrying out investigations at this time which we hope to have ready for presentation next year, to determine if such a reduction in fluorides may be expected over a long period of time. We expect our tests will be completed by the end of next summer and trust that we will gain greater knowledge of the fluoride problem in its entirety through this investigation.

### PERFORMANCE GUARANTEE

The plant was constructed under a performance guarantee bond written by a reputable bonding company, which insures its successful operation and maintenance of capacity for a period of five years after the plant is initially placed in operation. In so far as we know, this is again a first time, i.e., a municipal water softening plant constructed with guaranteed performance for a five year period.

ACKNOWLEDGMENTS

G. A. Youngberg & Associates, Inc. are Consulting Engineers on the plant, and the writer (Vice-President of the firm), is the designer and supervisor of construction. The writer wishes to express his appreciation for the coöperation received from PWA, the City of Sarasota, Florida, and particularly the Permutit Company, who set up the test plant and whose softening equipment is being used in the present plant under construction. This plant was placed in operation on April 15, 1937. The results of its operation have been very satisfactory.

# TRENDS IN ZEOLITE SOFTENING*

By Sheppard T. Powell

(*Consulting Chemical Engineer, Baltimore, Maryland*)

It is a recognized fact among water works engineers that municipal water softening is being widely adopted. Even with the present heavy tax burdens, many citizens are willing to accept the added burden of expense in order to enjoy the use of soft water. Up to 1929 more than one hundred water softening plants were in operation in this country. Of this number, more than 30 per cent were located in Ohio. Until that time the majority of such plants were designed to operate as chemical softeners. Since then, marked interest has been shown in the use of zeolites, either alone or in combination with miscellaneous chemical treatments.

It would be superfluous, and beyond the scope of the present address, to present an academic discussion of the value of municipal water softening. It is the author's sole purpose, here, merely to present a review of recent trends in zeolite softening of municipal water supplies, and to direct attention to the relative merits and demerits of such systems in comparison with water softening by chemicals.

It is assumed that the reader is familiar with the basic chemical reactions of base-exchange materials and that a review of the chemical phenomena involved in softening water by these substances is unnecessary. Such a discussion, therefore, has been avoided in order to devote the time allotment to the more pertinent phases of the subject as they are related to municipal water treatment.

Since the introduction of zeolite water softening into the country about 25 years ago, great strides have been made in the application of this process. All of the earlier installations were made for the conditioning of water for industrial uses and it was not untii about 8 years ago that zeolite water softening of public water supplies was undertaken. During the intervening years fairly rapid progress in the adoption of this type of treatment has occurred. A partial list of such systems is shown in table 1. By these data the rapid rate

* Presented at the Buffalo Convention, June 8, 1937.

TABLE 1

*Partial list of zeolite softening plants installed for the treatment of municipal water supplies*

| PLANT | DATE OF INSTALLATION | INSTALLED CAPACITY IN G.P.D. | HARDNESS OF RAW WATER, G.P.G. | HARDNESS OF SOFTENED WATER, G.P.G. |
|---|---|---|---|---|
| Florida: | | | | |
| Hollywood | Aug. 1936 | 1,500,000 | 17 | 5 |
| Sarasota | May 1937 | 1,000,000 | 65 | 6 |
| Illinois: | | | | |
| Ashland | Dec. 1936 | 42,000 | 32 | 5 |
| Clarendon Hills | July 1936 | 50,000 | 30 | — |
| Colchester | Oct. 1935 | 60,000 | 21 | 5 |
| Greenfield | July 1936 | 50,000 | 28 | 5 |
| Homewood | April 1935 | 500,000 | — | 5-10 |
| Jonesboro | 1937* | 216,000 | 22 | 5 |
| Monticello | March 1936 | 1,000,000 | 14 | 5 |
| Stonington | Nov. 1934 | 60,000 | 18 | 5 |
| Indiana: | | | | |
| Bicknell | May 1937 | 600,000 | 30 | — |
| Crown Point | July 1936 | 1,000,000 | 30 | 6 |
| Iowa: | | | | |
| Glidden | June 1936 | 60,000 | 23 | 5 |
| Kentucky: | | | | |
| Guthrie | 1937* | 110,000 | 40 | 6 |
| Louisiana: | | | | |
| St. Joseph | March 1937 | 60,000 | 14 | — |
| Michigan: | | | | |
| East Lansing | Jan. 1935 | 1,000,000 | 20 | 5 |
| Missouri: | | | | |
| Ste. Genevieve | 1936 | 120,000 | 30 | 10 |
| New Jersey: | | | | |
| South Orange | 1929 | 1,500,000 | 7.5 | 1 |
| New York: | | | | |
| Bethlehem | July 1935 | 118,500 | 11 | 5 |
| East Rochester | Sept. 1935 | 336,000 | — | 3-5 |
| Elba | June 1935 | 24,000 | 82 | — |
| Palatine | Jan. 1937 | 40,000 | 17 | 5 |
| Utica | July 1934 | 350,000 | 15 | 5 |

* Under construction.

TABLE 1—*Concluded*

| PLANT | DATE OF INSTALLATION | INSTALLED CAPACITY IN G.P.D. | HARDNESS OF RAW WATER, G.P.G. | HARDNESS OF SOFTENED WATER, G.P.G. |
|---|---|---|---|---|
| North Carolina: | | | | |
| Haw River | March 1933 | 30,000 | 19 | — |
| Ohio: | | | | |
| Berea | March 1936 | 500,000 | — | — |
| Cuyahoga Falls | Sept. 1936 | 3,000,000 | — | 5 |
| Elmore | Dec. 1934 | 50,000 | 63 | 4-6 |
| Grove City | Aug. 1935 | 42,120 | 35 | 5 |
| Lancaster | March 1933 | 1,260,000 | 22 | 5 |
| Mew Madison | April 1937 | 90,000 | 22 | 4 |
| Pemberville | Feb. 1935 | 60,000 | 23 | 4 |
| Plymouth | Aug. 1934 | 111,000 | 32 | 6 |
| Rittman | Aug. 1935 | 200,000 | 12 | 4 |
| Worthington | Feb. 1935 | — | 24 | 6 |
| Pennsylvania: | | | | |
| Coraopolis | Jan. 1936 | 1,000,000 | 12 | 3.5 |
| Somerset | May 1936 | 250,000 | 10 | 2 |
| Springdale | Oct. 1929 | 200,000 | 22 | 4 |
| Texas: | | | | |
| Belleville | July 1937 | 60,000 | 13 | 4-5 |
| Three Rivers | Dec. 1935 | 50,000 | 25 | 5 |
| Wisconsin: | | | | |
| Galesville | Aug. 1935 | 200,000 | 13 | — |
| Ontario, Canada: | | | | |
| Simcoe | Sept. 1936 | 200,000 | 16 | 5 |

of growth is indicated. This statistical record is of special interest, considering the fact that the period noted covers the depression years. The actual volume of water treated by the plants specified is shown in figure 1. The steep upward trend of this curve is indicative of the popularity and apparent successful performance of these systems. This is even more strongly demonstrated in figure 3, showing the wide-spread geographical locations where such systems have been established. With the return to normalcy a still more rapid development may be anticipated.

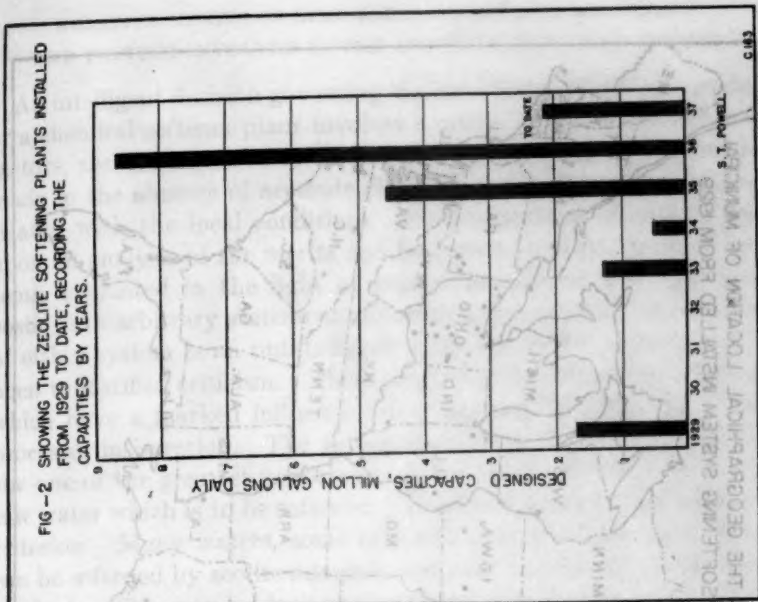



FIG – 2 SHOWING THE ZEOLITE SOFTENING PLANTS INSTALLED FROM 1929 TO DATE, RECORDING THE CAPACITIES BY YEARS.

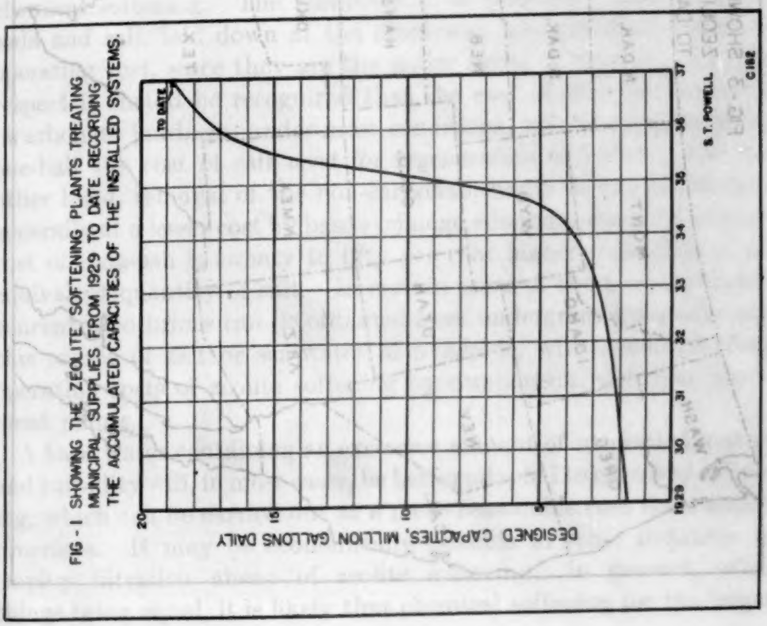



FIG - I SHOWING THE ZEOLITE SOFTENING PLANTS TREATING MUNICIPAL SUPPLIES FROM 1929 TO DATE RECORDING THE ACCUMULATED CAPACITIES OF THE INSTALLED SYSTEMS.

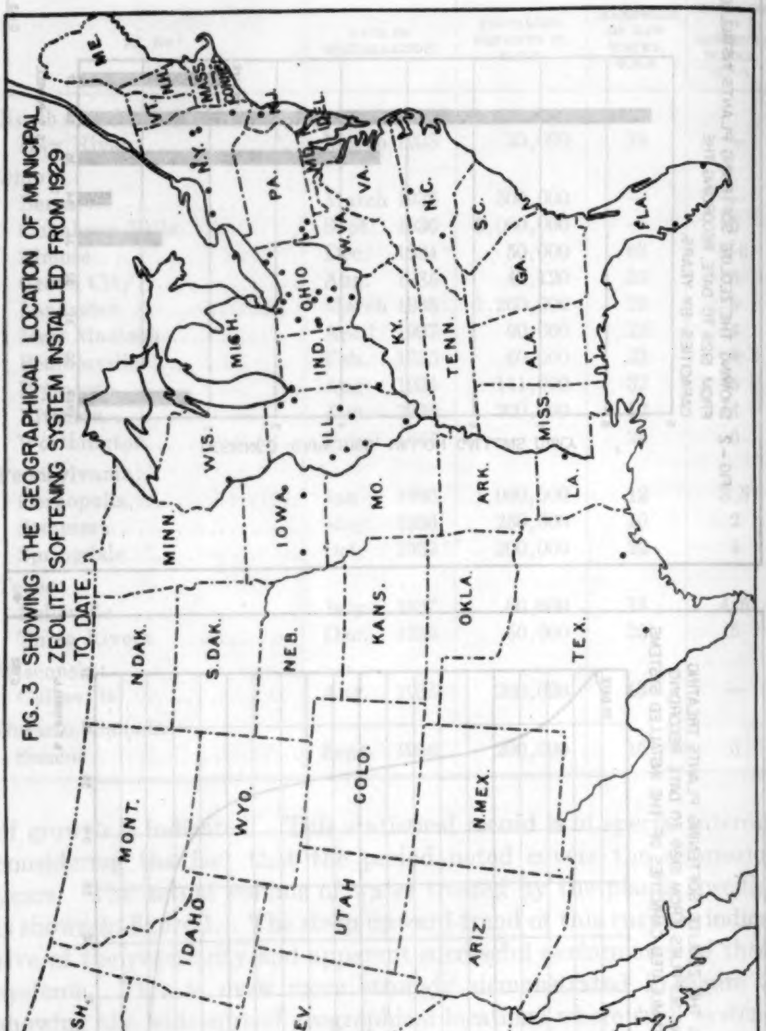



FIG. 3 SHOWING THE GEOGRAPHICAL LOCATION OF MUNICIPAL ZEOLITE SOFTENING SYSTEM INSTALLED FROM 1929 TO DATE

## RELATIVE MERITS OF LIME-SODA AND ZEOLITE SOFTENING AND FACTORS INVOLVED IN THE ADOPTION OF EITHER SYSTEM

An intelligent decision governing the installation of either a zeolite or a chemical softener plant involves a critical study of a number of factors, and a decision as to the choice of either plant should not be made in the absence of accurate chemical and engineering data correlated with the local conditions. Such a decision must also rest upon an analysis of the merits and limitations of both types of systems, evaluated in the light of capital investment and operating costs. An arbitrary statement advocating the adoption or rejection of either system is an unintelligent approach to the subject and is open to justified criticism. There are, however, certain basic factors which have a marked influence for or against the adoption of the processes in question. The principal deciding factor, and initially the one of the greatest importance, is the chemical character of the raw water which is to be softened. Hardness alone is an insufficient criterion. Many waters, some of which may be exceedingly hard, can be softened by zeolite minerals and yield a perfectly satisfactory product. This may be done as cheaply or even at a lower cost than chemical softening. The relative cost of chemicals, namely, lime, soda and salt, laid down at the plant site, has a marked effect on operating cost, since they are the major items of expense. In this respect it should be recognized that the cost of lime for removing bicarbonate hardness, under most conditions, will be approximately one-half the cost of salt used for regeneration of zeolite. On the other hand, removal of the non-carbonate hardness can be effected generally at a lesser cost by base exchange minerals, since the average cost of soda-ash is twenty to fifty per cent higher than that of an equivalent quantity of salt. In certain areas of the country highly concentrated brines can be obtained from underground deposits and this source of salt or sea-water, if available, will favorably effect operating costs of zeolite softening by comparison with lime treatment plants.

A hard water containing an excessive amount of suspended matter and turbidity will, in most cases, be better adapted to chemical softening, which can be carried out at a more reasonable cost than zeolite processes. It may be economically possible in some instances to employ filtration ahead of zeolite softening. In general, other things being equal, it is likely that chemical softening for the larger


*Courtesy of the Permutit Company*

FIG. 4


*Courtesy of the Permutit Company*

FIG. 5

municipal supplies will, in the future, predominate over combination filtration and zeolite softening.

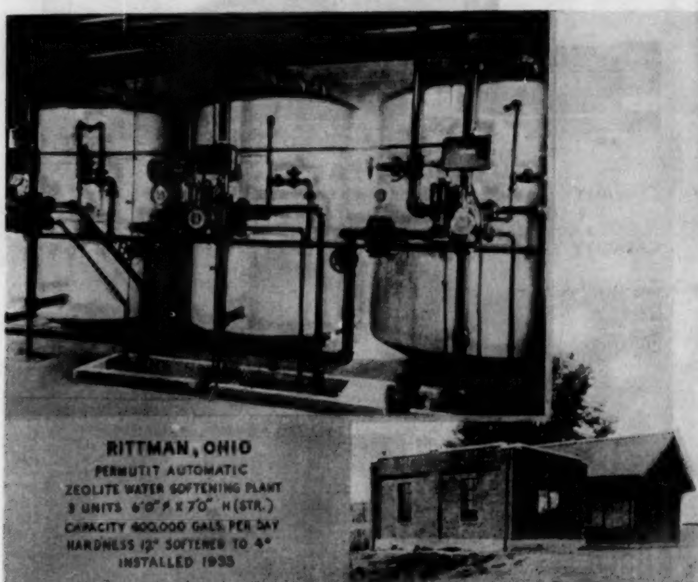
One of the most important, if not the principal, governing factors in deciding the relative merits of the two types of water purification equipment involves the cost of pumping. In cases where the existing pumping equipment is so designed that the water may be pumped


*Courtesy of the Permutit Company*

FIG. 6

directly through pressure zeolites, a decided advantage may be gained by the installation of this type of equipment in preference to chemical softening, which requires primary and secondary pumping. The handling and storage of chemicals is likewise important and should not be ignored. There is a general trend at the present time to provide for underground wet storage of brine for the regeneration of

zeolites. Where such installations are possible the storage and handling of the brine solution is much less costly than storage space and equipment which must be provided for lime and soda or other auxiliary chemicals which may be employed in addition to the softening reagents. The disposal of lime sludge from chemical softeners frequently involves a difficult operating problem. The writer has in mind a chemical softening plant which was designed for the disposal of sludge into the city sewers. Later such disposal was prohibited and it became necessary to haul all sludge, by means of tank wagons,




*Courtesy of the Permutit Company*

FIG. 7

to a distant point and dispose of the lime sludge on a relatively valuable land site. This case, although an extreme one, exemplifies the importance of one phase of softening plant control which is often given slight consideration. This is a matter of growing importance and it should receive the most careful analysis in the initial stages of development before making a decision as to the type of system to install.

There is a growing misconception that zeolite softening systems are fool-proof, especially those designed as automatic systems. Zeolite

units, if properly designed, require less supervision than do chemical softeners. This is especially true where the chemical composition of the raw water supply fluctuates widely. Notwithstanding the apparent advantage of zeolite systems, they are not fool-proof and are similar in this respect to many other types of mechanical and chemical equipment. They cannot be relied upon to function continuously without intelligent control. Lack of adequate supervision must sooner or later result in reduced plant efficiency and costly damage to the equipment.

## ACCEPTABLE HARDNESS OF MUNICIPAL WATER SUPPLIES

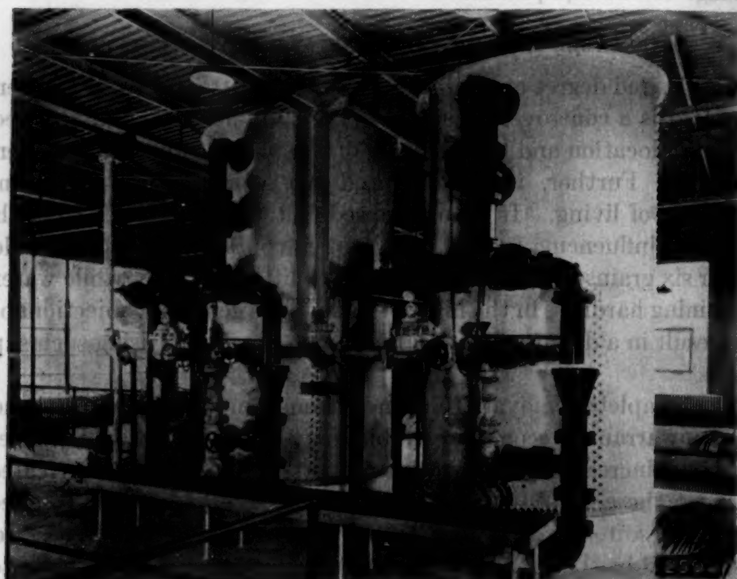
The desired degree of hardness in water after treatment in a softening plant is a controversial subject and is influenced greatly by geographical location and the hardness of the water supplied to adjacent territory. Further, it is influenced by social environments and standards of living. It is an accepted fact, however, aside from the foregoing influencing factors, that waters containing hardness under five or six grains per gallon are subject to little criticism, while waters containing hardness in the higher brackets are generally objectionable and result in a definite financial burden on consumers using such supplies.

The complete removal of hardness from a municipal supply is not only unwarranted economically but may cause objectionable aftereffects by increasing the corrosive properties of the waters so treated. It is for these reasons that few attempts have been made to effect complete softening. Where zeolite softening systems have been installed, it has been the practice to soften only a portion of the water to zero hardness and to mix the completely softened water with the hard water in various proportions so that the resulting supply delivered to consumers will contain a residual hardness between 3.5 and 7 grains per gallon.

## TYPES OF ZEOLITE

Base exchange minerals are generally classified as natural or synthetic and there are many grades of both types on the market. Zeolites, if improperly processed, may disintegrate and rapidly deteriorate. It is possible, however, to purchase from reputable manufacturers, good grades of either material. If the raw water passing through such mineral is properly adjusted, satisfactory service is assured. The principal difference between the two types of

zeolites is their softening capacity, between regenerations, per cubic foot of mineral. In the past, the average softening capacity of the natural or green sand zeolite was approximately 2800* grains per cubic foot. Properly manufactured synthetic or gel zeolites are capable of removing from 8000 to 12,000 grains per cubic foot and in some cases even higher exchange values have been obtained. It is obvious that when the high capacity zeolites are used, the size of the containers will be less than when the green sand is used. The grains of green



*Courtesy of International Filter Company*

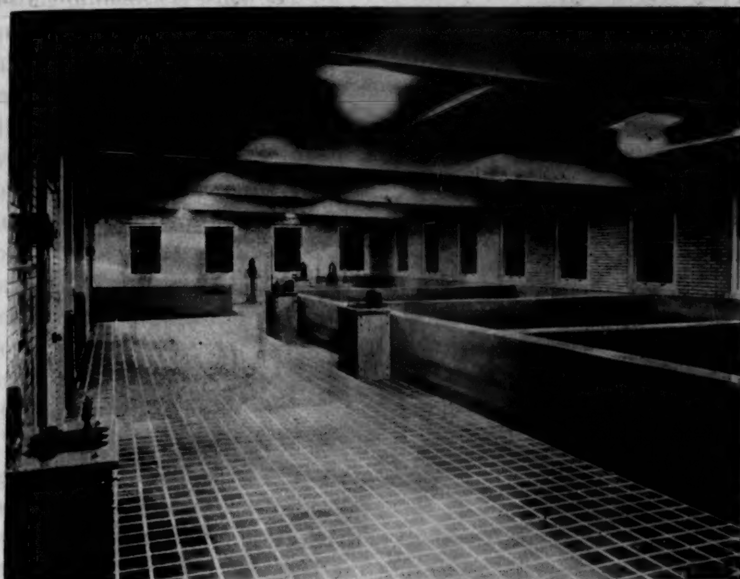
FIG. 8. TWO INFILCO 84-INCH CRYSTALITE SOFTENERS IN ZEOLITE SOFTENING PLANT AT SPRINGDALE, PA.

sand are very much finer than those of the gel type of mineral, so that the loss of head through the former type will generally be much greater than through the latter.

The most important item in the operating costs is the salt. An effort should be made, therefore, to regenerate the zeolite mineral with the smallest quantity of brine per unit of hardness removed. It is commonly assumed that the regeneration of zeolites requires one-

* Recently green sand zeolites have been prepared having much higher softening capacity.

half pound of salt per thousand grains of hardness (calculated as calcium carbonate) removed. In some well-operated plants, and under favorable conditions, a salt consumption lower than the figure quoted has been possible. At a number of plants the salt consumption has been materially reduced and a consumption of as little as 0.35 pounds per thousand grains of hardness removed has been possible without reduced exchange efficiency. In some cases extreme reduction in salt consumption has been effected. One operator reported .2 pound of salt per kilograin of hardness removed. This

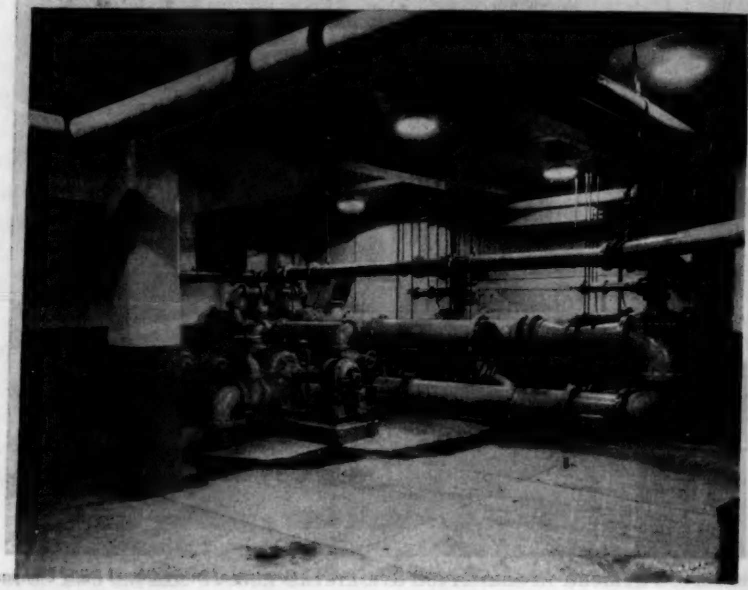


*Courtesy of International Filter Company*

FIG. 9. GRAVITY FILTERS IN THE ZEOLITE SOFTENING PLANT AT LANCASTER, OHIO

is an unusual condition and should not be adopted for design purposes. For municipal service, zeolites of high exchange value are desirable, since the higher the exchange value, the less frequently backwashing and regeneration of beds will be required, and the smaller the units will be. It is good practice to design green sand softeners for a rate of from 4 to 5 gallons per square foot of softening area per minute, although higher rates have been used successfully under some conditions. With the coarser synthetic type of mineral, rates of 8 gallons or higher are permissible without detrimental effect.

As previously pointed out, finely divided green sands result in a relatively higher head loss during a softening period than will occur with coarser gel type minerals. This fact is of the greatest importance in deciding the relative merits of the two systems, since the loss of head through a unit of the pressure type will have a marked effect on pumping charges and the resulting cost of operation. Hoover,* in 1933, in describing the operation of the Lancaster, Ohio, plant equipped with a gel type zeolite, recorded the wash water and salt savings over a green sand plant to be an amount sufficient to pay for the synthetic zeolite bed in slightly more than one year.



*Courtesy of International Filter Company*

FIG. 10. PIPE GALLERY AND PUMP FOR TWO 12-FOOT BY 15-FOOT INFILCO CRYSTALITE UPFLOW SOFTENERS AT LANCASTER, OHIO

## THE EFFECT OF ZEOLITE SOFTENING ON INDUSTRIAL WATER SUPPLIES

There is no economic justification for a municipality to supply completely softened water to industrial users for their various requirements. Notwithstanding this accepted premise, municipal authorities or operators of privately owned plants are morally ob-

* C. P. Hoover, *Public Works*, **64**, 9 (1933).

ligated to give some consideration to the users of water for industrial purposes. Softening of public water supplies, in the majority of cases, results in a proportionate saving to industrial consumers, many of whom require soft water. On the other hand, conditioning public water supplies, even though adequately softened within the zone of economic responsibility, may result in unforeseen difficulties. The speaker refers specifically to the conversion of scale-forming solids into sodium salts, especially carbonates and bicarbonates. Water supplies so treated may become so unbalanced as to cause operating difficulties, when the water is used for boiler feed purposes or for other industrial processes. Where radical changes in the chemical characteristics of water occur, it is the responsibility of the municipality to so advise the industries that the necessary corrective measures may be taken by those affected and operating damages thereby avoided. In a recent investigation conducted by the speaker, the chemical characteristics of the municipal water supply, after treatment, were so changed as to render the supply potentially objectionable from the standpoint of embrittlement of boiler steel. Such conditions are much more likely to happen where the supply is treated by zeolite softening than when chemical processing is employed. This is a potential danger which has been given little or no consideration by designing engineers, but which, under some conditions, may constitute industrial problems of major importance.

The earlier type of municipal softeners were similar to those designed for industrial uses. More recently designers of large-size units have embodied desirable features of control similar to those employed in filtration plants, such as hydraulically controlled valves, rate of flow controllers, operating tables, and other auxiliary equipment. A number of plants built recently have been made largely automatic, by providing equipment for automatic regeneration and backwashing at predetermined time cycles. Such appliances tend to eliminate the personnel errors and to this extent have merit. On the other hand, there is danger of depending too much on automatic devices for the complete elimination of desired supervision. Such criticism, however, may be directed not only to the particular type of apparatus involved in this discussion, but is an inherent danger in all automatically-controlled equipment. Reference is made here to the potential danger of defective operation and the basic principle of control, rather than specific criticism of any special type of automatic control devices.

## COST OF ZEOLITE SYSTEMS

The cost of zeolite plants compared with other water purification systems may be expected to fluctuate widely. However, the capital investments and operating costs of a number of existing plants may be of interest. In table 2 are given the fixed and operating costs of 7 plants. The average cost of softening per grain of hardness removed per MGD was $5.59. The wide variation in fixed and operating charges is, of course, influenced by the size, type and design of the plant, cost of labor, salt, and a number of other items. The highest figures are not typical because of the inclusion of abnormal cost items

### TABLE 2

*Giving the capital investment and annual operating charges of municipal zeolite softening systems*

| Plant | Designed capacity of softener, G.P.D. | Capital investment | Annual fixed charges | Total annual operating costs* | Total costs per M.G. | Green sand or synthetic | Hardness of raw water | Residual hardness of softened water delivered to consumers | Softening costs per grain hardness removed per M.G. | Ratio of soft to hard water | Date installed |
|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 2,000,000 | $65,000 | $5200 | $13,080 | $35.80 | Syn. | 7.6 | 3.5 | 8.73 | — | — |
| B | 60,000 | 9,500 | 87 | 1,750 | 160.00 | Syn. | 23.0 | 5.0 | 8.89 | — | — |
| C | 1,000,000 | 21,000 | 1575 | — | 61.20 | Gr.S. | 10.5 | 2.5 | 7.65 | 75% | 1934 |
| D | 272,000 | 33,000 | 4100 | 9,110 | 91.10 | — | 27.8 | — | — | — | 1931 |
| E | 76,000 | 16,000 | 1200 | — | 121.10 | — | 51.0 | 4.0 | 2.58 | — | 1934 |
| F | 2,000,000 | 72,000 | 5400 | — | 61.34† | — | 23.0 | 4.0 | 3.23 | — | 1935 |
| G | 120,000 | 8,500 | 1200 | 3,283 | 49.44‡ | — | 30.0 | 10.0 | 2.47 | — | 1936 |

* Including fixed charges.

† Estimated.

‡ Not including building, land and miscellaneous accessories.

peculiar to the given plant, or because of the small size of the plant. It will be noted, however, that these figures are, in general, comparable with fixed and operating charges required for softening by lime or by lime and soda water softening systems. In most cases the figures quoted herein do not include cost of land, engineering and certain intangible items of expense.

It is difficult to predict accurately the extent of future developments of municipal softening by these processes. There is ample evidence, from the data submitted, to justify the prediction that they will be employed even more extensively in the future than during the

past. The reason for such a prediction is that the successful performance of existing plants now in operation and developments occurring in the art have demonstrated their serviceability and economical performance. In some instances local conditions will militate against such plants, and chemical softening plants may be better adapted and less costly to build and operate than base exchange plants.

### ENGINEERING FACTORS TO BE INVESTIGATED

It is impossible to list all the engineering factors which should be investigated and evaluated before arriving at a decision giving preference for either type of system under consideration. In general, the data listed below will be found of value and should be given consideration, in the order of listing, by engineers before reaching a conclusion as to the relative merits and adaptability of such plants.

1. Chemical characteristics of the raw water and fluctuation in dissolved and suspended solids.
2. Influence of pH value of the raw water on the disintegration of zeolite mineral and the cost of chemicals required for pre-treatment to correct such conditions.
3. Extent of sewage pollution of raw water and its effect on the efficiency and cost of operation of either system.
4. Extent of presence of iron and manganese and relative cost of removal by either system.
5. Space required for proposed plant.
6. Cost of land.
7. Cost of electric energy.
8. Cost of wash water for backwashing zeolites and softeners.
9. Necessary pumping head for primary and secondary pumping.
10. Cost of lime and soda-ash.
11. Possible re-use of lime and employment of by-product chemicals.
12. Cost of salt and availability of natural brines from underground sources or from sea-water.
13. Cost of water for regeneration and washing of brine from beds.
14. Cost of aeration or other pre-treatment ahead of softening systems.
15. Sewer facilities and cost of sludge removal from chemical softeners, versus disposal of brine waste from zeolites.
16. The necessity for recarbonation of chemically softened water to inhibit deposition of incrustations in the distribution system.

17. Cost of pre-filtration as opposed to filtration following chemical softening.

18. The operating costs of gravity and pressure softening.

19. Study of the relative corrosiveness of the softened water from a chemical softener and a zeolite system.

20. Wages of common and skilled labor and influence of these costs on manually or automatically controlled systems.

21. Other relative merits of automatic and manually controlled systems.

22. Relative merits of green sand and synthetic zeolites, which may influence fixed and operating costs.

## CONCLUSION

It has been the author's purpose merely to present existing trends and development in zeolite water softening for municipal use. As previously stated, the value of such systems has already been demonstrated and they are now beyond the experimental stage. The subject, therefore, requires thoughtful consideration of water works engineers, and the economics of municipal softening may no longer be adequately evaluated without giving consideration to this form of treatment.

# ORGANIZATION AND PREPARATION OF WATER WORKS TO MEET MAJOR CATASTROPHES

## PREPAREDNESS PLAN FOR SAN FRANCISCO

By N. A. Eckart

(*General Manager, Water Department, San Francisco, Cal.*)

*This and the three following papers comprised a symposium at the Buffalo convention on June 10, 1937. The general topic "Organization and Preparation of Water Works for Major Catastrophes" was discussed with reference to earthquakes, conflagrations, floods and tornadoes.*

The ability of a water works system to function in a major catastrophe will depend, primarily, upon the fundamental design and construction of the system, and, secondly, upon the organization and training of the operating personnel so that they may carry on under the emergency conditions which may arise.

Preparedness to meet such a catastrophe, whether it be in the form of a great fire, a flood, drought, a tornado or an earthquake must begin with the engineering design, particularly as to the location of the key elements, with regard to areas of hazard which might be involved in a catastrophe, and to the type and character of construction of those elements.

Flexibility is of prime importance, and by this we mean the ability to maintain service at a given point or area through several combinations of pipe lines and equipment,—this may involve providing spare units, auxiliary power supply, local storage, and duplicate installation of important mains or structures in widely separated locations, with proper interconnections and facilities for isolating ruptured mains, etc.

Experience gained in any catastrophe is invaluable in redesigning or extending the system so as to eliminate the weak spots and potential causes of failure and increase the dependability of the system to meet a repetition of the emergency which in most instances can be almost certainly depended on to recur with ever greater force.

Unless the system has been designed with due regard to the special conditions which may have to be met in a catastrophe, the efforts of even the best trained operating personnel may be wholly thwarted in maintaining service or in quickly restoring impaired or disrupted service. On the other hand, an experienced and trained organization is essential if one is to take advantage of the flexibility of the system in maintaining or re-establishing service in an emergency, even with a well designed and constructed system. In fact the more extensive and more flexible the system the greater is the need for organization and training of the personnel to utilize the facilities to the best advantage.

While this discussion involves reference to disaster preparedness in general, our water department program is designed, primarily, with the earthquake hazard in mind.

### LESSONS OF 1906 EARTHQUAKE

Following is a brief summary of the conclusions of a Committee of the American Society of Civil Engineers on the Effects of Earthquakes on Water Works structures:

1. Avoid as far as possible locating important water works structures on threatening geologic faults.
2. Well built earth dams deserve confidence.
3. Gravity concrete dams can withstand severe shocks without damage.
4. Distribution reservoirs, pumping machinery, tanks and stand pipes well designed and with good foundations will withstand shocks sufficient to destroy most buildings.
5. Pipe lines or conduits when intersected by a plane of large movement must fail and such locations should be avoided when practicable. In distributing systems important supply pipes should be located along lines located for stability and areas liable to serious disturbance should be segregated by gate valves.
6. Gate valves should be generously distributed throughout the pipe system, these should be properly maintained and adequately recorded. Important supply lines should be in duplicate along widely separated routes. Ample supply of water should be maintained in close proximity to centers of use. Substantial design should be adopted for structures subject to shock.

In addition to the above, we consider that in important pump stations where continuous operation is paramount, the station should

be equipped with steam or diesel power for prime movers and not be dependent for operation upon transmitted electric power.

Since 1906, at least, the engineers responsible for the development of the San Francisco system have been earthquake conscious and this has been reflected in the subsequent design and improvements, although noting the splendid character of construction and the manner in which the dams, pumping stations and other structures withstood the earthquake one can not help but feel that the earlier engineers had in mind the possibility of heavy seismic disturbances in this region.

Without going into detail, we may note some of the more important features of the present system which make for its dependability in a catastrophe:

1. A dependable source of supply at Hetch Hetchy with capacity far in excess of present requirements.

2. Storage capacity in the local bay area equivalent to 1000 days' usage, one-half of which is on the Peninsula within fifteen miles of San Francisco.

3. Four separate transmission mains located with due consideration of earthquake faults, separated as far as practicable and with inter-connections, delivering water by gravity from these reservoirs to receiving or distributing reservoirs within the City.

4. Capacity of distributing reservoirs within the City of 320 million gallons (including new units under construction). These reservoirs are located at strategic points throughout the City with interconnecting pipe so that water from the higher level reservoirs can be delivered into the system served by the lower ones.

5. In general, there are two outlet supply pipes from these reservoirs separated in location as far as practicable.

6. The number of secondary feeders connecting the grid system with the supply mains from the distributing reservoirs has been materially increased, with due consideration to providing for bypassing around areas which may be cut out in case of local damage.

7. Changed operating conditions following the completion of the newer transmission mains providing increased gravity delivery have permitted the abandonment of four of the old steam pumping stations, which while they had successfully withstood the 1906 earthquake nevertheless afforded a degree of hazard in case of a repetition of a disturbance of that magnitude.

8. Additional gate valves are being installed in the grid system to reduce the size of the gated sections.

It will be noted that the features mentioned accord generally with the recommendations of the Earthquake Commission. While it has not always been possible from a practical point of view to conform fully to these recommendations, particularly as to complete separation of pipe routes and avoidance of bad ground, it is felt that the volume of storage provided in the City and the ability of the operating organization to handle the emergency will minimize the effect of an earthquake disturbance even of the magnitude of 1906.

## GENERAL PLAN FOR DISASTER PREPAREDNESS

The preparedness plan of the Water Department is coordinated with and forms part of the general disaster preparedness plan of the Municipal Government. This general plan was developed by a special committee appointed by the Mayor and with the full coöperation of the military officials, Red Cross, heads of municipal departments and various public utilities and civic agencies.

*Authority for plan:* Section 25 of the Charter of the City and County provides as follows:

> "In case of a public emergency involving or threatening the lives, property or welfare of the citizens, or the property of the city and county, the mayor shall have the power, and it shall be his duty, to summon, organize and direct the forces of any department in the city and county in any needed service; to summon, marshal, deputize or otherwise employ other persons, or to do whatever else he may deem necessary for the purpose of meeting the emergency. The mayor may make such studies and surveys as he may deem advisable in anticipation of any such emergency."

The Charter prescribes rather definite contract procedure for the purchase of supplies and for the execution of public works, but provides—

> "In an emergency, provided an actual emergency be declared by the Board of Supervisors to exist, and when authorized by resolution of said board, any public work or improvement may be executed in the most expeditious manner."

Pursuant to this the Board of Supervisors under the "Contract Procedure" ordinance has defined certain emergencies under which

department heads may move directly in making repairs and handling what may be termed limited emergencies as distinguished from a general disaster requiring approval of the Commission head or the Mayor and the Controller, or

> "If the emergency does not permit such approval to be obtained before work is commenced said approval as hereinbefore mentioned shall be obtained as soon thereafter as it is possible to do so."
>
> "The department head concerned shall notify the Controller immediately of the work involved and the estimated cost thereof."

*The General Plan:* No attempt is made in this paper to present any more than a bare outline of the general plan to indicate the relation of the Water Department plan to the whole.

Under the plan the Mayor or the acting Mayor is the responsible and directing head, with the Chief Administrative Officer, and Director of Public Works acting as his immediate aides and succeeding him in authority in that order in case of absence or disability. Provision is made for appointment of an advisory council by the Mayor. An emergency staff is established as follows:

| | |
|---|---|
| Law and Order | Chief of Police |
| Fire and Rescue | Chief Engr. of Fire Dept. |
| Health and Sanitation | Director of Public Health |
| Utilities | Manager of Utilities |
| Streets, Sewers & Bldgs. | Director of Public Works |
| Communications | Chief of Dept. of Electricity |
| Intelligence and Coordination | City Engineer |
| Personnel (Police Only) | The Sheriff |
| Rescue | |
| Personnel (other than Police) | Registrar |
| Transportation | |
| Materials and Supplies | Purchaser of Supplies |

Provision shall be made for at least two alternates to every key position.

General Headquarters are established at the City Hall, if that building is safe, otherwise at Jefferson Square (Jefferson Square is an open public park in which is located the Central Fire Alarm Station, a low thoroughly fire and earthquake proof building). Located at General Headquarters will be the Mayor, the Advisory Council, his Emergency Staff and the key men of all divisions.

The plan prescribes in general the organization and duties of the several divisions headed by the members of the emergency staff.

The Manager of Utilities, as executive officer of the Public Utilities Commission, represents that body in the emergency staff. He has charge of Water Supply—low pressure, Street lighting, Street Railways, Air Port. Provision is made for coöperation of privately-owned power and railway utilities and shops fabricating pipe, etc.

With reference to Water Supply, the general plan provides—

> "An adequate supply of water for fire protection and domestic purposes shall be maintained at all times. The plans for an organization to provide such supply, will be completed by the General Manager and Chief Engineer of the Water Department."

WATER DEPARTMENT PREPAREDNESS PLAN

In an earthquake of major intensity great destruction can occur within a few minutes without previous warning. Some of the immediate factors affecting operation of the water supply system may be:

To cut off all electric light and power;
Cut off all communication by telephone and telegraph;
Disrupt all street car service;
Throw down building walls, blocking streets;
Start fires throughout area due to broken gas pipes, broken flues, short circuit wires;
Rupture water, sewer and gas mains;
Put pumping plants and water treatment plants out of commission.

Any plan of preparedness must anticipate these conditions so that in spite of them service can be maintained or rapidly restored.

Such plan calls, first, for proper organization of the personnel in the various divisions, with line of authority clearly established, so that in the absence or disability of the division head or of a key man under him, there will be a definite succession of responsibility and authority.

Probably next in order of importance is the advance designation of headquarters and stations to which the various employees shall report upon knowledge that the disaster is of serious magnitude. Alarm indicating such disaster may be given by siren, radio, telephone or other means, or the situation may be sensed by personal observation. Several alternate headquarters should be designated.

to be successively occupied in the event the first named is not safe or tenable.

Standing orders applicable in an emergency should detail individuals to take definite action without further notice. Thus—employees who have automobiles assigned are instructed to pick up other named individuals at definite addresses and convey them to the established headquarters; reservoir keepers, pump station attendants, patrolmen, watchmen, and those regularly assigned to special operating and maintenance duties, are instructed immediately to inspect the properties or structures under their care and report conditions by telephone or messenger or, if necessary, in person to the established division headquarters; engineer in charge of auxiliary pumping plant is to arrange to get up steam and prepare to start pumps. As our plan has been worked out, the four main distribution districts in San Francisco are subdivided into a total of fifteen sub-districts, key men are assigned to each of the four main districts with one or two gatemen and helpers together with a runner with automobile for each of the sub-districts. It is the duty of the runner to cover the district and report to the key man all major leaks immediately upon discovery, and minor leaks within the hour. The key man will direct the gateman and crew in shutting off sections that are leaking badly; the key man shall, through the runner, report conditions and action taken to the superintendent's headquarters. Standing orders call for continued operation of central pumps unless the supply to the pumps is cut off.

All employees in each division not specifically detailed to special assignments are directed to proceed to the headquarters for their respective divisions as best they can, likewise those assigned to cars are to proceed to headquarters by any available means if not picked up within an hour after the quake.

Upon receipt of reports from the various key men throughout the city the superintendent or alternate in general charge is directed to map his course of action and design crews to make repairs at the various points of damage throughout the City in the order of their importance.

*The Peninsula Division* includes the several large storage reservoirs in San Mateo County and the connecting conduits and the transmission mains leading to San Francisco as well as a section of the Hetch Hetchy Bay Crossing Pipe bringing water to Crystal Springs. Here, standing orders to individual caretakers, patrolmen,

etc. call for immediate inspection of structures, including chlorinating apparatus, reading of Venturi meters to check excessive flows and report to Millbrae; operator at Crystal Springs pumps which lift water from Crystal Springs reservoir to San Andres storage reservoir is instructed to shut these pumps down. Based on reports received, repair crews will be dispatched to make repairs. The water production engineer will assume direction and charge of repairs to major structures, with assistance of men from engineering department. All Peninsula employees not given individual assignments will report to Division headquarters. Report conditions to San Francisco headquarters.

*Alameda Division:* Standing orders provide for closing gates at Irvington (west portal of Hetch Hetchy Tunnel Aqueduct); to reduce flow in Bay Crossing Pipe Line to about 3 m.g.d. (sufficient for requirements of attached consumers); close outlet gates Calaveras reservoir, if open; order water cut off at Hetch Hetchy if there is no line of communication with San Francisco; report conditions to Millbrae headquarters; arrange for any necessary patrol at various structures and hold other employees, including those of Agricultural Department, available and ready to report to Peninsula Division headquarters upon orders to that effect. Due to the larger water storage available on the peninsula, except for small local requirements, the entire Alameda Division can be taken out of service for an indefinite period.

*Water Purification Division:* Each man has definite assignments—providing for inspection of chlorinating plants in San Francisco, including making any minor repairs necessary to permit functioning, or shutting down, and making note of necessary parts, or work to be done—and arrangement for emergency chlorinating equipment. They must be prepared to handle chlorination of pipe lines as restored to service after breakage and repair.

General orders call for the accounting and water sales forces not otherwise specifically assigned, to report first to main office and protect records if the building is threatened, and thereafter to report to the corporation yard to assist in handling reports, keeping records, distributing material and performing other duties as assigned.

The plan requires maintenance of a good stock of pipe, fittings, valves and other material. Arrangements have also been made with a large pipe fabricating company for use of welding equipment

and shop facilities in such emergency. Location of all pipe, valves and connections, throughout the City together with pressure zones, are shown on maps on 400-ft. scale cut into sections of convenient size and bound into books. These books are kept up to date and are at all times in possession of the gatemen, general foreman, superintendent, etc. A portable chlorinating machine is maintained for emergency use as well as a stock of hypochlorite of line in measured packages for chlorinating pipe lines before placing into service.

# ORGANIZATION AND PREPARATION OF WATER WORKS TO MEET MAJOR CATASTROPHES
## CONFLAGRATIONS*

By Henry E. Halpin

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It is well known that water works operators are imbued with the spirit that "the show must go on." We have come to consider that there is a direct obligation on the water works operator to maintain continuous service, ample to meet domestic and fire requirements of the community served. This the operator recognizes and is prepared to do under all ordinary circumstances. Much study of the best operating practice, an intimate knowledge of the pumping plant, the source of supply, and the distribution system and an unselfish devotion to his trust are in this respect the working tools of the good operator.

Beyond the ordinary emergencies for which plans have been made is the conflagration. Few communities are without the possibility of a severe sweeping fire which could cause extensive damage and which would tax to the very extreme all the resources of the water works.

The term "conflagration" is frequently used in two ways, to mean a severe localized fire or secondly, one of a sweeping or far reaching nature. For the purpose of this discussion, we will limit our consideration more to the type of conflagration which extends over a considerable area and destroys a number of buildings. The causes of conflagrations of this type can be narrowed down to a combination of two or more of the following circumstances. The first-named, fuel, must always be present. First, the origin of a fire in a section containing much combustible construction; second, failure or inadequacy of the water system; third, limitations in equipment or personnel of the fire department; and fourth, action of the elements favorable to the spread of a fire. In this last group would come

* Presented at the Buffalo convention, June 10, 1937.

extreme dry and hot spells, extreme cold, tornado or other windstorm, floods and earthquakes. This type of fire, requiring the delivery of vast quantities of water throughout a widespread area, would create a severe concentrated demand much in excess of any ordinary peak demand.

The National Board of Fire Underwriters in adopting the Standard Schedule for Grading Cities and Towns with Reference to Their Fire Defenses and Physical Conditions has established as a measure of the ultimate capacity of a system under conflagration conditions, that the system should be able to deliver, in addition to domestic demands, the fire flow set forth in the table, at any pressure down to 20 pounds, except that a minimum of 10 pounds is permissible in high value districts having well distributed hydrants of satisfactory size and design. The schedule requires that the system should be capable of maintaining the fire flow requirements for a duration of ten hours during a period of five days maximum consumption for cities over 2500 population and for five hours for cities under this population.

| POPULATION | REQUIRED FIRE FLOW FOR AVERAGE CITY | POPULATION | REQUIRED FIRE FLOW FOR AVERAGE CITY |
|---|---|---|---|
| | *gallons per minute* | | *gallons per minute* |
| 1,000 | 1,000 | 28,000 | 5,000 |
| 2,000 | 1,500 | 40,000 | 6,000 |
| 4,000 | 2,000 | 60,000 | 7,000 |
| 6,000 | 2,500 | 80,000 | 8,000 |
| 10,000 | 3,000 | 100,000 | 9,000 |
| 13,000 | 3,500 | 125,000 | 10,000 |
| 17,000 | 4,000 | 150,000 | 11,000 |
| 22,000 | 4,500 | 200,000 | 12,000 |

Over 200,000 population, 12,000 gallons a minute with 2,000 to 8,000 gallons additional for a second fire.

In reviewing an analysis by the National Fire Protection Association of the principal factors contributing to conflagrations in the United States and Canada between 1900 and 1925, it appears that in thirty-seven out of one hundred cases inadequacy of the water distribution system was a major factor. A more recent analysis by the Inspection Department of the Associated Factory Mutual Fire Insurance companies of twenty-three conflagrations occurring between 1914 and 1934 indicates that in eleven cases inadequacy of

the water supply was a major factor, while in one case the entire water supply failed due to the burning of power lines which supplied the pumping units.

There are instances on record where during conflagrations the fire demand has exceeded the National Board's requirements. Examples of such instances are the fire at Nashua, New Hampshire, in 1930; at Fall River, Mass., 1928; Salem, Mass., in 1914; and Chelsea, Mass., in 1908. The demand in these instances exceeded the National Board's requirements by 100 to 200 percent. There are several instances on record where it was possible to meet the fire demand because the conflagration occurred at a time of low domestic consumption; for instance, on a Sunday, as was the case at Nashua, N. H. The possibility of quickly assembling motorized equipment from relatively distant cities for use at conflagrations further increases the likelihood that the demand will more frequently exceed the recognized standard.

Conflagrations are not frequent occurrences. Fire prevention work, a more general installation of automatic sprinklers, and the adoption of improved types of construction are minimizing the number, yet it behooves every operator to give serious consideration to the possibility of such an occurrence in his community and to develop definite plans to mobilize all the facilities at his command to augment and conserve his supply and deliver to the required location the greatest possible quantity of water.

The problem of making available ample supply of water to care for conflagration requirements divides itself into two parts. First, the long time planning and reconstruction of the system to eliminate any deficiencies and, second, the planning of measures to be adopted under emergency conditions. In this respect each community will have a different problem and on each superintendent or department head will rest the responsibility for the proper organization and preparation.

Study should first be made of locations where hazardous conditions exist, where the construction is of a type which would not resist exposure to fire and would be conducive to the spread thereof, where high winds are frequently experienced and where, due to congestion of structures, difficulties might be experienced in successfully fighting fires. Records show that wood shingles have contributed to the spread of many sweeping fires. Of primary importance is the prevention of interruption to pumping service. Every

precaution should be taken to protect structures housing essential units from exposure. Structures of combustible construction might well be protected with automatic sprinklers. Good practice would require that power lines enter pumping plants underground. A careful survey should be made to determine the adequacy of the gridiron system to supply such locations as are considered hazardous a quantity of water at least equal to the amount set forth as desirable under the National Board's schedule. Supporting mains to such locations should be so designed that the required quantity of water will be available not only at the locus of the hazard, but at such surrounding points as will enable the fire department to combat, with some degree of success, any fire originating thereon.

Physical connections of ample size should be made to all neighboring supplies and data as to the quantities and pressures available from such connections should be developed and tabulated. Studies of pressures and of the normal direction of flow in various sections of the distribution system should be made either by use of the pitometer or large-sized meters and as a result of these studies plans should be developed and recorded so that in case of a severe peak load such as would be caused by a conflagration, the systems could be gated off so as to secure the best results. In some instances this could be accomplished by temporarily feeding outlying sections from neighboring systems or low service reservoirs, having pressures insufficient to be of value at a fire of major proportions. In other cases the neighboring supplies might be used to augment the normal supply. Definite plans should be established to secure the best results for every hazardous location.

A possible deficiency in supply can sometimes be offset by the construction of elevated tanks or standpipes. Such structures should be located so that the additional water impounded therein would be available through ample-sized mains to the point of hazard.

In locations where a potable water is filtered for the purpose of removing microscopic organisms or color, or where there are water softening or iron removal plants, with the approval of the State Health Authorities, by-passes could be utilized to increase temporarily the supply available.

A definite outline should be made of the responsibilities of various members of the water works organizations which should be established to combat the emergency resultant from a conflagration. The officer in charge should be entirely familiar with the conflagration

survey, with the various connections to neighboring systems and their value and limitations and with the most satisfactory methods of utilizing the same to their full advantage. He should direct the operation of gates throughout the system so as to furnish with a maximum of pressure, the greatest quantity of water possible and should keep in close contact with the officer in charge of the fire fighting organization, should direct the operation of the pumping plant, and of the emergency crew. The police department telegraph system or radio system may furnish a satisfactory means of communication by which the department head can keep himself informed as to the operation of his pumping station and the amount of water available in reservoirs. It is here assumed that the officer in charge of the water department would be present at a conflagration and would be established with or near the chief of the fire department. The operators of pumping stations and filter plants should be thoroughly instructed as to the manipulation of their plant in order to gain the greatest advantage under emergency conditions. Definite outline of procedure leisurely made, may prevent serious and costly confusion under stress.

Prompt notice of fires is important. Arrangements should be made to receive fire alarms automatically at water department headquarters and pumping stations. Good coöperation should exist with the fire department and a waterworks emergency truck should respond at least on a second alarm and thus be on the scene of a possible conflagration early.

The emergency crew should consist of the most reliable members of the water works organization, should be equipped with complete records of the locations of gate valves, including valves on large industrial and fire services, gate operating equipment, gate boxfinders, pumps, etc. Such a crew should be in constant attendance to execute the orders of the officer in charge, to prevent loss of water through broken services or mains and abandoned hydrants, and to render to their superior officer complete reports as to the pressure and suitability of the water supply.

It occasionally happens that where the distribution mains are of inadequate size, and replacements have not been effected, some advantage can be gained by pumping with a fire engine pump as a booster from a main, around a closed gate, and into the same main, thereby reducing or offsetting normal friction loss. A knowledge of the adequacy of the distributing mains would enable the

officer in charge to suggest to the fire department, locations where any advantage could be gained through such a use of engine pumpers.

Ingenuity on the part of the water works official will result in greatly increased efficiency which may be the saving feature in the control of a bad situation. It is interesting to note that at the Chicago Stock Yards fire on Saturday, May 19, 1934, effective use was made of the radio to appeal to the public to limit their use of water during the period of the conflagration. The public water department reported that, probably as a result of the broadcast, the pumpage from several pumping stations was less than normal and that the pressure at the pumping stations and in the mains near the stock yards actually increased 5 to 10 lb. during the fire, despite the use of about 55,000 gallons of water per minute from the public mains. The public would ordinarily be responsive to an appeal of this sort which should be sent out as soon as it has been determined that there is a need to conserve water or to increase pressures.

It is the writer's opinion that the number of unsatisfactory performances of waterworks during conflagrations can be materially reduced by careful studies of hazardous locations, the development of adequate mains in and around such danger spots, the use of elevated tanks or standpipes to sustain flow to such districts, the development of a program of operations to augment the supply either from within the system or from neighboring supplies, and to conserve in various ways all available water for use in extinguishment of fires.

# ORGANIZATION AND PREPARATION OF WATER WORKS TO MEET MAJOR CATASTROPHES
## FLOODS*

By E. A. Munyan

*(Consulting Engineer, Cincinnati, Ohio)*

The 1937 flood disaster in the Ohio valley was particularly interesting to anyone operating water properties and provided many lessons which should be remembered for years to come. Steps should be taken immediately to eliminate the possibility of their recurrence.

One of the outstanding items of importance to anyone operating in a flood disaster is that of determining in advance—and as much in advance as possible—just what is happening on the upper stages of the river and what you can expect in your individual property or series of utilities. One must rely on weather predictions to determine how much rainfall may come or how much has fallen in a given period and how it will affect you. Such facilities are usually provided by the U. S. Weather Bureau but their facilities are quite limited and proved entirely inadequate in the 1937 Ohio valley flood. Their predictions were too low, due probably to the fact that some of the communication systems were out of order and due to the fact that they only have men on duty at certain times of the day and night. It is hoped that this deficiency will be remedied before the next flood as the flood crest predictions were lower than some of us to whom better private communication systems were available, knew they would be.

When flood waters are rising a constant watch must be maintained and any item of danger or particular interest must be recorded and acted upon immediately. In fact, one must look ahead and plan for any emergency which may arise. If they do arise, you can meet them and if they do not arise, you have lost nothing at all.

Another outstanding item noticed as the flood arose, was that some operators have no emergency plans available such as are in

* Presented at the Buffalo Convention, June 10, 1937.

plan and in use in some of the west coast cities. Even at this time, months after the flood, many of the cities in the Ohio valley have not planned any such emergency organization. Some attempts have been made but have not been completed. It is very, very easy to forget a flood or other disaster after it has passed by and then some persons wait for the next emergency to arise.

When the flood waters arose and when some of the water pumping stations in the Ohio valley were in danger of going out of operation, it was thought by some that all water customers should be advised to fill all their bath-tubs, wash-tubs, jugs, etc. with water ahead of time and while operations were normal and while an abundance of water was available. This would have meant a lot of private water storage available for the water emergency. This was suggested to some of those in charge and it was further suggested that advertisements be placed in the papers asking the water customers to store water. The suggestion met with disfavor, others feeling that it might scare the water customer. In my opinion, it is much better for our customers to be scared than to have them without any water from the water systems for 6½ days as happened in some cases and for longer periods in others.

When the emergency did arise and the disaster was upon us all, however, everyone jumped in and did his best to meet the emergency and we all came through alive and safe. Some of the chances taken, however, were terrible to say the least and were not good operating practice. There is some evidence that raw river water was turned into certain water mains without notification to the customer and if such can be prevented in the future, we will certainly be benefited.

It is hoped that something will be done immediately to remedy the entire water situation as far as flood areas are concerned and to provide better engineering methods and better operations in such emergencies. Those in the water operating departments did their best but they are all held down by rules, regulations, ordinances, lack of funds, and this condition must be changed if the emergency water operations are to be improved.

## RECOMMENDATIONS

The following recommendations are suggested for what they are worth from one who has passed through several floods in the Ohio valley in the past ten years in various utility operations.

(1) Greater storage reservoir capacity must be available.

(2) Storage capacity must be located well out of the high water area.

(3) Water pumping plants must be located well above high water.

(4) An additional emergency supply must be available.

(5) A scheme of organization properly coördinated with other community functions, must be not only planned but tried out from time to time in manner similar to other fire and emergency drills.

(6) All distribution and transmission systems must be so planned and operated that flood areas can be sectionalized at any time of disaster.

There are many other items of good engineering practice which may be done to assure continuance of supply. There is plenty of work available which should be planned to take care of such emergencies well ahead of their occurrence.

My recommendation to the association is that a committee be organized by the A. W. W. A. to collect data and publish plans for water works emergencies and that when such plans are completed that they be given wide publicity.

I also recommend that each and every city or private water company look into its emergency operating plans and if none are available, someone should draw them up and see that they are perfected just as soon as possible.

# ORGANIZATION AND PREPARATION OF WATER WORKS TO MEET MAJOR CATASTROPHES*

## TORNADOES

By Albert R. Davis

(*Superintendent, Water Department, Austin, Texas*)

To anticipate the catastrophe of a tornado and make preparations for what might happen, at first, may seem to be a very indefinite task. Tornadoes strike in the most unexpected places and their vagaries are beyond human mind to foresee.

The American Legion has organized a "preparedness for catastrophes" committee, and in numerous cases, it's members have stepped in and proven to be of inestimatable aid in controlling the situations that have developed. This suggests the idea that if we give some thought to being prepared in the water works field for an emergency, we may be able to act more quickly and effectively than if we had never considered emergency preparation.

### SOURCES OF SUPPLY

There are three vital points in any waterworks system, namely; Supply, Pumping, and Distribution. The water supply of any town will be either impounded, from wells, or from rivers. In case of a tornado, impounded supplies would suffer least because, as a rule, dams are built with sufficient strength and ruggedness to withstand even a tornado. In numerous cases, dams are at such an elevation that the water flows to the treatment plant, thereby eliminating the necessity of a pump station. The well supply is usually attended by a pump of some description, unless it happens to be an artesian well. Where pumping is required, we should ask ourselves: How long will the town survive without the use of this well? How long will it take to repair any damage that might occur to the power source? Is the power source single or double? Would standby equipment be justified?

Where the supply is taken from a river, the situation is even worse. Low lift pumps are required to raise water to the treatment plant, and the same questions may be asked in regard to these pumps, plus a few additional ones such as: Is the pump house secure against tornadoes? Is the screen tower capable of withstanding one? What

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emergency power could be utilized on short notice for pumping? In case of pump breakage due to falling of heavy weights, what standby pumps are available? Do you know where you can get a substitute pump on short notice? Do you have an old pump that could be rushed into service in emergency? Do any of your neighboring towns have equipment that they could loan you for a short time? In most cases, pump stations located for river service are constructed to withstand floods and, consequently, are strong enough to withstand wind storms.

## FILTER PLANTS

Visualize, if you can, a twister taking off the headhouse and the filter building; the equipment for feeding chemicals wrecked by falling structure. What temporary measures could you adopt to keep treatment of water going? Are all of your chemical supplies stored in the same building? Do you have a safe place for employees to go to in such cases?

## PUMPING PLANTS AND BOOSTER STATIONS

Pumping plants are probably the most unreliable part of the entire water works system. There are normally three sources of power for pumping: steam, electric motor, and gas or oil engines. Where steam is used, the stack from the boiler is an easy prey for a strong wind. Is it possible for you to operate without the use of these stacks?

Where electric energy is used, are the lines leading to the pump station on poles, or underground? What is the probability of the electric system failing completely in such cases? What standby equipment do you have for use in case of power failure?

Gas or oil engines, where used alone in single or duplicate units, are probably less vulnerable to storms, yet there are precautions to be taken in this case. These engines, if housed in a reinforced concrete structure, might be considered safe against any wind storm.

Booster pump stations are where the fun comes in. It is common practice where a booster pump station is used to have an elevated water tower. These towers are an easy prey for a tornado. How many of you have seen an elevated tank sprawled out on the ground? It is a pitiful sight. Would you be able to operate with the pumps at the booster without the use of the tank? Would it be feasible to install a pump of the proper size to supply the territory without the

use of the tank? As a rule, booster pumps are located in such a way that the destiny of an entire city would not be materially affected, but only the section supplied by the booster, and while this section is less important, yet it should not be considered lightly. The safest plan for an emergency is to ask the same questions as apply to the main pump station.

### THE DISTRIBUTION SYSTEM

The worst situation as far as the distribution system is concerned centers around the possibility of a group of fire hydrants being broken off and the valves controlling them being covered by debris; telephones out of service; men trained for repair work out seeing what has happened; repair equipment blocked by wreckage. What would you do?

This situation offers the opportunity to make the key suggestion of the entire program of preparedness,—*Training and Discipline of the Entire Water Works Personnel.* A book could be written on training alone, and still leave volumes unsaid, but, briefly, men who know how to do their jobs well and to accept the responsibility for them will be the biggest asset of any waterworks in case of disaster. As a part of the training of your personnel, at your weekly or monthly meetings outline the above situation and ask them the simple question "What would you do?"

How many people in the average city know how to protect themselves from a cyclone or a tornado? A few oldtimers may have a dugout, but not many. I, for one, know of a safe place where I shall take my family if I see the tornado in time. And as to seeing a twister, allow this suggestion: If it appears to be moving, stay where you are. It will pass to one side. If it appears to be standing still, then you'd better move.

### WATERWORKS STRUCTURES

Preparedness for tornadoes should start in the designing period and—where possible—with some additional cost the structures should be designed to meet any wind condition. In many places where buildings are designed to withstand floods, no additional cost will be necessary for wind protection, and where possible, new waterworks structures with some additional cost should be designed to meet the maximum wind condition.

## CONCLUSION

In making preparation for tornadoes or any disaster; first, design buildings, where possible, to withstand disaster from natural sources; second, train personnel in what will be expected of them in case of emergency disaster; third, make a list of all standby equipment that could be used in case of an energency, and urge neighboring cities to prepare a similar list. Then form an agreement with each other that in case of necessity, the equipment so listed is available.

# DOSAGE OF CHLORINE IN AMMONIA-CHLORINE TREATMENT OF WATER*

By Prof. M. L. Koshkin
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The bacteriological examination of water is the surest and final test of the correct dosage of chlorine; this method is constantly applied to control chlorination. However, since this method requires considerable time—24 to 72 hours, the development of more rapid chemical methods appears desirable.

Active chlorine can be determined by various methods: according to Ellms and Hauser (1) with ortho-tolidine; according to Olszewski (2) with benzidine, and by Bruns (3) iodometric method, etc. Of these the ortho-tolidine and iodometric methods are the ones most frequently used. The ortho-tolidine method, while being very sensitive, has certain drawbacks. For instance, the same yellow color as with chlorine is produced when manganese compounds are present; the reaction is slowed up to some extent in the presence of chloramines (4) and the sensitivity of the test decreases with increasing concentrations of chlorine. The test also cannot be used with colored waters.

The iodometric method, although less sensitive, was chosen for the experiments presented in the following pages because it is free from these drawbacks. However, the iodometric method also has its weaknesses and these must be taken into account. When using this method (titration of the free chlorine in the presence of acid) the nitrites and ferric salts liberate iodine from potassium iodide and when the initial amount of chlorine is insufficient this will cause incorrect results. In order to avoid these errors Skopintzev and Varfolomeyeva (5) have proposed buffering the solution at pH 4.6 with an acetic acid buffer solution (Walpole) and then bring the pH of the water up to 5.0–5.5. When nitrites are present the results may be in error due to the acid added in the titration for chlorine.

* Contributed record of research.

Further errors are possible when acid is added to the water due to the splitting of certain substances which have a tendency to produce more or less stable compounds with free iodine and chlorine. Froboese (6) has shown that in the titration of free residual chlorine in the determination of the chlorine demand ("chlorbedarf" according to Bruns) this figure is greater in samples titrated in the presence of acid and the difference in the respective values may be considerable, amounting to 0.1–0.4 mg. per litre in the experiments of Froboese. The same author has also demonstrated that when different acids are added in titrating residual chlorine, the results are not the same; under the same conditions hydrochloric acid causes more chlorine to be liberated than does acetic acid. Froboese used the term "abspaltbares Chlor" (chlorine capable of being split off) for the chlorine liberated upon addition of acid. According to his findings there are waters that yield such chlorine, while other waters do not. Thus the chlorine demand of the same water may be different according to whether determination was made with or without acid; the chlorine demand found when using acid being less than the value obtained without the use of acid and the difference will be equal to the amount of half-bound chlorine (as we will agree to call chlorine liberated at the addition of acid). Harold (7) has also pointed out the significance of acid in titration of free chlorine.

The chlorine demand likewise changes under the influence of several other factors. We shall only consider those that have a bearing on the questions considered in this study.

The initial dosage of chlorine to a large extent influences the chlorine demand. The experiences of Schmidt and Mühlenbach (8) have shown that the greater the initial amount of chlorine, the higher the chlorine demand of the water. The pH of the water also exercises a great influence on the chlorine demand. Ammoniation of water may especially influence the chlorine demand of the same; our experiments (9) with different substances proved the great importance of ammonia in this respect.

The usual iodometric method of determination of chlorine demand (at room temperature) differs but little if practiced according to the directions recommended by various authors. It consists in the addition of a certain amount of chlorine (usually 1–2 mg.) to 100 cc. or 1 litre of the water to be tested and after the water has been left in contact with the chlorine dose for a certain time (10 to 30 minutes) in the dark, 2 to 10 cc. of a 10 percent KI solution and 5 to 10 cc.

of an 8–10 percent HCl or $H_2SO_4$ solution are added to it; then the chlorine that had been left unabsorbed is titrated with a N/200 solution of $Na_2S_2O_3$ on addition of 1 cc. of 1 percent starch solution. The difference between the initial amount of chlorine added and the unabsorbed residue is usually termed chlorine demand. This latter is most often determined according to the method of Bruns which is similar to the method we have just described. Bruns determines the chlorine demand in 1 litre of water by adding 1–2 mg. chlorine to the same, permitting a contact of 10 minutes in the dark and then titrating the chlorine left unabsorbed with N/100 $Na_2S_2O_3$, with 10 cc. of a 10 percent KI solution, 10 cc. of 8 percent hydrochloric acid and 1 cc. of a 1 percent solution of starch.

TABLE 1

*Decrease of chlorine demand of water under the influence of ammonia*

| DISTILLED WATER WITH ADMIXTURE OF | AMOUNT OF SUBSTANCE | DOSAGE OF $NH_3$ | CHLORINE DEMAND | DECREASE IN CHLORINE DEMAND |
|---|---|---|---|---|
| | | *mg. per litre* | *mg. $Cl_2$* | *percent* |
| Phenol | 10 mg. | | 11.262 | |
| | | 1.384 | 0.942 | 91.7 |
| Hay infusion | 2 cc. | | 1.132 | |
| | | 0.412 | 0.389 | 65.6 |
| Meat-peptone broth | 2 cc. | | 0.687 | |
| | | 0.608 | 0.604 | 12.1 |
| Glucose | 10 mg. | | 0 | |
| | | 1.03 | 0 | 0 |

These methods of determination of the chlorine demand are quite unsuitable for finding the dosage of chlorine in ammoniated water. In water-purifying plants using ammoniation the dosage of chlorine is chosen empirically and then verified by bacteriological count.

We made it our task to work out a chemical method for the determination of chlorine dosage in ammoniated water. In so doing we were guided by the chlorine demand and took into account all the facts referred to above. We chose the iodometric method as the one that best suited our purpose.

The first factor that must be considered in working out the method was the change in chlorine demand of the water under influence of

ammonia. As was shown by our experimental investigations, the chlorine demand of different substances changes in different ways under the influence of ammonia; with some substances the chlorine demand is diminished to a fraction of its former value, while with others it is decreased but slightly and with still others it is not diminished at all. The data in table 1 (9) bring this out more clearly.

As the chlorine demand is of paramount importance in determining the dosage of chlorine, it follows that this determination (according to the methods we have used, that is the determination of the dose of absorbed $Cl_2$) must be made in the presence of a certain amount of ammonia or a compound of ammonia. Thereby the value of the chlorine demand is considerably altered and the dose of $Cl_2$ found by way of titration more closely approaches the actual demand.

TABLE 2

*Chlorine demand with different amount of initial chlorine dose*

| DISTILLED WATER | AMOUNT OF SUBSTANCE ADDED | INITIAL AMOUNT OF Cl | RESIDUAL Cl | ABSORBED Cl |
|---|---|---|---|---|
| | *cc.* | *mg. per litre* | *mg. per litre* | *mg. per litre* |
| With peat decoction | 2 | 3.0 | 1.35 | 1.65 |
| | 2 | 2.47 | 1.20 | 1.27 |
| | 2 | 1.75 | 0.60 | 1.15 |
| With horse serum | 0.2 | 2.45 | 1.20 | 1.25 |
| | 0.2 | 1.46 | 0.69 | 0.77 |

The second fact that must be taken into account when using our method of determination of chlorine dosage is the amount of the initial chlorine dose, the determination of which has an important influence on the chlorine demand. Our experiments confirming those of Schmidt and Muhlenbach are tabulated in table 2 (10). These findings prove that with increasing initial chlorine dose the chlorine demand rises. Notwithstanding the proofs of the influence of the initial chlorine dose on the chlorine demand, the methods now in use do not take this fact into consideration. Without considering the initial chlorine dose it is impossible to forecast an exact chlorine dosage. The same waters may show different values of chlorine demand depending upon the initial chlorine dosage used in determining the chlorine demand.

After many trials we adopted the following principle which excludes the error caused by variations in the initial chlorine dose. As the residual chlorine is dependent upon the initial dose, the latter can be regulated so as to produce a constant value of residual chlorine. There is no strict proportion between the amount of the initial chlorine dose and the amount of residual chlorine for, in changing the dose, the chlorine demand is altered likewise. For each water sample, however, a definite initial chlorine dose will correspond to a certain residual of unabsorbed chlorine. We found empirically that the dose of chlorine leaving 0.3–0.4 mg. residual chlorine per litre produces a good bactericidal effect. We determined the chlorine demand while the residual chlorine constantly remained at 0.3–0.4 mg. per litre, creating equal conditions also for determination of the dosage of chlorine. The chlorine demand is determined by means of repeated analyses with various initial amounts of chlorine. At the same time the initial amount of chlorine is chosen in such a way that the residual unabsorbed chlorine should amount to 0.3–0.4 mg. per litre, and we consider the amount that is absorbed by 1 litre of water under these conditions the correct chlorine dose.

The third factor of importance in determining the chlorine dosage is the addition of acid in the titration of the residual. In our study of the influence of pH on the chlorine demand of ammoniated water (11), we have shown that the chlorine demand rises while the pH diminishes. Evidently this depends on the formation in alkaline solution of hypochlorites that are less active than chlorine. When alkali is in great excess, the chlorine demand of the water becomes very low at ordinary temperatures, although the water may contain a large amount of chlorine-absorbing substances.

When there is alkali in the water the reaction between the former and chlorine may proceed by way of formation of hypochlorite, according to the following equation:

$$HOCl + KOH \rightleftharpoons KOCl + H_2O$$

or

$$2\,KOH + Cl_2 \rightleftharpoons KOCl + KCl + H_2O$$

The further course of the reaction may then be accompanied by formation of chlorates

$$2\,HOCl + KOCl \rightleftharpoons KClO_3 + 2\,HCl$$

The reaction with iodine set free during the application of the iodometrical method may proceed in an analogous way with formation of iodates:

$$3I_2 + 6KOH \rightleftharpoons 5\,KI + KIO_3 + 3H_2O$$

Still, evidently more chlorates than iodates are formed, for chlorine is present in a high concentration and the contact with the substances contained in the water is longer. The reaction between chlorine and iodine follows the afore-mentioned equations when the water has an alkaline reaction. But when the water is acidified the reaction goes on to the end while chlorine and iodine are set free from the chlorates and iodates.

Therefore, chlorination of water which has an alkaline reaction produces chlorates which are considerably less active than chlorine and do not liberate iodine from KI in the absence of acid. We know that in determining the chlorine demand by the iodometric method, titration of the residual unabsorbed chlorine is performed with addition of acid by which active chlorine of the chlorates is set free. If this titration (which we term back titration) is carried on without the addition of acid, then the chlorine of the chlorates will remain bound. Owing to this, the residual will decrease while the chlorine demand will rise in the same measure. Thus the chlorine residual for alkaline water found by iodometric titration in the presence of acid will be less than that determined in the absence of acid. If the chlorine dose found by titration in the presence of acid (i.e. the diminished dose) is added to alkaline water, a certain part of the active chlorine will serve to form chlorates and thus the bactericidal action of the chlorine dose added will be lower. In water of great alkalinity a large amount of chlorates may be formed and the amount of active chlorine may prove so small that no bactericidal effect whatever will result. In water with acid reaction no chlorates and hypochlorites are formed. But nevertheless, even such water as is not intensely acid under natural conditions may contain substances which form unstable compounds with chlorine and from which active chlorine is liberated under influence of strong acid. The formation of chlorates and the presence in the water of substances liberating absorbed chlorine on addition of acid, i.e. the formation of half-bound chlorine during chlorination of water, create such conditions that determination of chlorine dose may result in an amount of chlorine with an insufficient bactericidal effect. Ordinarily a dose

determined by titration in the presence of acid will be found insufficient only when the water is highly alkaline; if the reaction of the water is slightly alkaline or acid, the dose of chlorine determined by means of the usual iodometric method produces a fair bactericidal effect; the dose of chlorine according to this determination will be sufficient (sometimes even too great (Bruns)), and the transformation of a certain amount of chlorine into half-bound $Cl_2$ will in most instances have no untoward effect on the bactericidal action.

Quite a different situation is created when the chlorine dose is titrated in ammoniated water. Here the chlorine demand is greatly reduced and consequently the dosage of chlorine is also decreased. Titration in the presence of ammonia yields the smallest dosage, especially when acid is added to the water, which likewise diminishes the amount of active chlorine. The dosage determined in the aforementioned manner may prove insufficient as regards its bactericidal effect, especially when the water has an alkaline reaction. It also must be borne in mind that introduction of ammonia likewise diminishes the chlorine demand. In actual plant practice, unlike in the test tube, no acid is added and therefore the conditions are not the same as when determining the chlorine demand in the laboratory.

In order to find out the importance of acidification in determining the active dosage of chlorine as judged by the chlorine demand, we made several experiments. The first series was made on samples of 100 cc. of distilled water, after its pH had been brought to different values by addition of HCl or NaOH. Organic substances added to the water were hay infusion (50 gms. hay in 1 litre water heated to 120° for 20 minutes and then filtered) and meat-peptone broth as used by bacteriologists.

Chlorination was performed with a weak solution of chlorine; ammoniation with $(NH_4)_2SO_4$; the ammonia was introduced into the water 10–20 seconds before the chlorine. In this series of experiments we tried to determine the significance of acid in the back-titration of residual chlorine. We titrated the residual chlorine, contrary to the usual iodometric method, first without acid. Then we added acid and again titrated the chlorine thus liberated. Table No. 3 contains data regarding the chlorine demand of distilled water with the addition of hay infusion and broth. The residual chlorine was titrated in two steps, first without and then with acid. Data on the determination of half-bound chlorine are also given.

All samples were treated with the same chlorine dose. The experi-

ments showed that the amount of half-bound chlorine is greater in the dilute broth than in the dilute hay infusion. In the samples with differing doses of hay infusion the amount of half-bound chlorine hardly changed, while in samples with broth the amount of chlorine rose together with the amount of broth. Evidently the broth contains more compounds capable of forming unstable combinations with chlorine from which the latter is set free by acid. Table 3 likewise shows that when titration is performed with acid the chlorine demand is lower than without acid, the decrease corresponding to the amount of half-bound chlorine that is set free upon the addition of acid.

Analogous data are found in titrating residual chlorine, with and without acid in water with different pH values.

TABLE 3

*Half-bound chlorine in water with hay infusion and broth*

| | AMOUNT OF SUBSTANCE | INITIAL AMOUNT OF CHLORINE | RESIDUAL CHLORINE BEFORE ADDITION OF ACID | HALF-BOUND CHLORINE | CHLORINE DEMAND | |
|---|---|---|---|---|---|---|
| | | | | | Without acid | With acid |
| | | *mg.* | | *mg.* | | |
| Hay infusion | 0.5 | 0.936 | 0.617 | 0.068 | 0.319 | 0.251 |
| | 2.0 | 0.936 | 0.343 | 0.034 | 0.593 | 0.559 |
| | 3.0 | 0.936 | 0.274 | 0.068 | 0.662 | 0.594 |
| Broth | 0.5 | 0.936 | 0.511 | 0.068 | 0.425 | 0.357 |
| | 2.0 | 0.936 | 0.254 | 0.103 | 0.682 | 0.579 |
| | 3.0 | 0.936 | 0.151 | 0.137 | 0.785 | 0.648 |

This experiment was performed with distilled water with the addition of hay infusion or broth and 1 mg. $(NH_4)_2SO_4$. The chlorine demand was determined with and without the addition of acid.

As is shown in table 4, the chlorine demand determined by means of titration with addition of acid was different from the chlorine demand found without acid, the difference becoming greater with the rise of pH. When pH equalled 7.0 the chlorine demand without acid was twice as great as with acid; when pH equalled 8.1 it was three times greater without than with acid. This experiment showed the rôle of acidification during back-titration very clearly. It also shows the significance of the choice of one or the other method of determining chlorine dosage for the treatment of water. Also we could prove that the amount of half-bound chlorine increased with

the rise of the initial dose of chlorine. The experiment was made in ammoniated water with pH 7.7 (when the rise in half-bound chlorine accompanying the rise of initial dose of chlorine is more evident).

As is shown in table 5, raising the chlorine dose from 0.5 to 2.0 increased the amount of half-bound chlorine from 0.120 to 0.298 mg.

These experiments showed that the amount of half-bound chlorine may be considerable. In ammoniated water it may even be greater than the amount of absorbed chlorine.

TABLE 4

*Chlorine demand of ammoniated water with pH of various values*

| pH | $(NH_4)_2SO_4$ | INITIAL AMOUNT OF CHLORINE | CHLORINE DEMAND WITHOUT ACID | CHLORINE DEMAND WITH ACID |
|---|---|---|---|---|
| | *mg.* | | | |
| 4.0 | 1 | 0.821 | 0.406 | 0.342 |
| 7.0 | 1 | 0.821 | 0.438 | 0.219 |
| 7.6 | 1 | 0.821 | 0.454 | 0.203 |
| 7.9 | 1 | 0.821 | 0.518 | 0.187 |
| 8.1 | 1 | 0.821 | 0.534 | 0.187 |

TABLE 5

*Half-bound chlorine and initial amount of chlorine*

| HAY INFUSION | $(NH_4)_2SO_4$ | INITIAL AMOUNT OF CHLORINE | RESIDUAL CHLORINE BEFORE ADDITION OF ACID | HALF-BOUND CHLORINE | CHLORINE DEMAND | |
|---|---|---|---|---|---|---|
| | | | | | Without acid | With acid |
| | *mg.* | *mg.* | | *mg.* | | |
| 0.5 | 1 | 0.5 | 0.249 | 0.121 | 0.251 | 0.180 |
| 0.5 | 1 | 1.0 | 0.508 | 0.209 | 0.492 | 0.283 |
| 0.5 | 1 | 1.5 | 0.870 | 0.269 | 0.630 | 0.361 |
| 0.5 | 1 | 2.0 | 1.316 | 0.298 | 0.684 | 0.386 |

This makes it easy to understand the necessity of performing back-titration of chlorine residuals without the addition of acid. The change in the method ruled out instances of deficient bactericidal action of chlorine that had been observed when chlorine dosage was determined by titration in the presence of acid.

Moreover, titration without acid excluded the possibility of liberation of iodine from potassium iodide in the presence of nitrites. It must be emphasized, however, that we propose iodometry without acid-addition only for determining the active dose of chlorine in

ammoniated water. The method of determining the proper dosage of chlorine for ammoniated water is the following. In a 2 litre bottle with a glass stopper we first pour 1 litre of the water under investigation; then the dose of ammonia or ammonia salts that has been decided upon is added. The amount of chlorine that may approximately be wanted is then carefully stirred in and permitted to stand for 10 minutes in the dark. Ten cubic centimeters of a 10 percent solution of potassium iodide and 1 cc. of 1 percent starch is then added. The solution is then titrated without the addition of acid with a N/200 thiosulphate solution. This determination is repeated with various initial amounts of chlorine until the residual unabsorbed chlorine will equal 0.3–0.4 mg. per litre. We regard as active chlorine dosage the amount of chlorine absorbed under these conditions, i.e. the difference between the initial amount of chlorine and this amount of residual chlorine.

TABLE 6

*Bactericidal action of chlorine in water containing meat-peptone broth*

| pH | $(NH_4)_2SO_4$ | Dose of chlorine | | Count of colonies | | | | |
|---|---|---|---|---|---|---|---|---|
| | | With acid | Without acid | 15 minutes | 30 minutes | 1 hour | 2 hours | 24 hours |
| | *mg.* $NH_3$ | | | | | | | |
| 5.4 | 0.52 | 0.397 | | 44 | 16 | 0 | 0 | 0 |
| 5.4 | 0.52 | | 0.413 | 32 | 20 | 0 | 0 | 0 |
| 7.0 | 0.52 | 0.338 | | 391 | 122 | 43 | 10 | 0 |
| 7.0 | 0.52 | | 0.446 | 58 | 36 | 1 | 0 | 0 |
| 8.2 | 0.52 | 0.238 | | Many | Many | Many | Many | Many |
| 8.2 | 0.52 | | 0.509 | 418 | 64 | 11 | 0 | 0 |

A number of experiments were made with the chlorine dosage determined by titration in this way. The principal criterion for the acceptability of the new method of determining chlorine dosage is the question whether such a dose is sufficient for a bactericidal effect. One of many experiments to elucidate this question is represented in table 6. It was made in 1 litre of tap water with the addition of 0.5 cc. broth and 25 million of a 24 hour culture of *E. Coli* (according to standard). First of all the pH of the water was adjusted; then the substances required for the experiment were added and lastly the necessary chlorine dosage was found according to our method; as control we used a dose of chlorine determined by titration in the

presence of acid. In this experiment $(NH_4)_2SO_4$ was added because $NH_4OH$ could have altered the pH of the solution. The dose of ammonia was found according to method described in one of our previous papers (12). Table 6 indicates that at pH 5.4 the bactericidal action of a dose of chlorine titrated with and without acid gave nearly the same result; at such a low pH the dose of chlorine titrated with and without acid differed but very slightly (the difference amounted to 0.016 mg.). With the pH at 7.0 the difference between the chlorine dose titrated with acid and without it was greater (0.108 mg.). As could be foreseen the bactericidal action of the dose of chlorine determined without acid was greater than that of the dose determined with acid; nevertheless the latter also had a

TABLE 7

*Bactericidal action of chlorine in water with gelatine and hay infusion*

| | Amount of organic matter per liter | Dose of chlorine: With acid | Dose of chlorine: Without acid | Dose of $(NH_4)_2SO_4$ | Count of colonies — Duration of contact between water and chlorine: 30 minutes | 1 hour | 2 hours | 4 hours |
|---|---|---|---|---|---|---|---|---|
| Gelatine | 5 mg. | 0.108 | | 0.44 | Very many | 7,656 | 1,133 | 150 |
| | 5 mg. | | 0.266 | 0.44 | 1,062 | 15 | 0 | 0 |
| | 5 mg. | | 0.266 | None | Very many | 500 | 157 | 21 |
| Hay infusion | 0.5 cc. | 0.207 | | 0.44 | Very many | 2,159 | 426 | 2 |
| | 0.5 cc. | | 0.431 | 0.44 | 566 | 1 | 0 | 0 |
| | 0.5 cc. | | 0.431 | None | 1,203 | 230 | 3 | 1 |

considerable bactericidal effect. At pH 8.2 the dose of chlorine titrated with acid proved inefficient and when water treated with such a dose was examined it yielded a rich growth of colonies, while the dose determined without acid gave a good bactericidal effect. The difference between both doses amounted to 0.271 mg.

An analogous experiment was made with tap water and gelatine (5 mg. per 1 litre of water) and with water containing hay infusion (0.5 cc. per litre). The pH of the water was 7.6. The experiments presented in table 7 also prove that a chlorine dose titrated with addition of acid does not have a sufficient bactericidal effect, even in ammoniated water. A dose of chlorine titrated under the same conditions but without addition of acid has a fair bactericidal action.

Further, these experiments demonstrate that a dose of chlorine without ammonia has a considerably weaker bactericidal effect than the same dose would have in ammoniated water.

The pH value and the dose of chlorine exercise an opposite influence upon the bactericidal effect. A rise of the pH lowers the bactericidal action of the chlorine; raising the dose of chlorine it is possible to compensate for this decrease. The dose of chlorine as determined according to the chlorine demand with the addition of acid diminishes when the pH rises; when the pH value is high this dose is inefficient as regards bactericidal action. On the contrary, the values found for the chlorine dose titrated without acid increases while the pH value rises. This increase in the dose compensates for the diminished bactericidal action of chlorine when the pH rises and thus the determination of chlorine dose without acid is a more correct method.

TABLE 8

*Residual chlorine various periods after chlorination*

| | DOSE OF CHLORINE | RESIDUAL CHLORINE | | | | |
|---|---|---|---|---|---|---|
| | | After 1 hour | After 2 hours | After 4 hours | After 6 hours | After 24 hours |
| | | *mg.* | *mg.* | *mg.* | *mg.* | *mg.* |
| With acid | 0.246 | 0.165 | 0.192 | 0.062 | 0.030 | Traces |
| Without acid | 0.326 | 0.234 | 0.145 | 0.113 | 0.094 | 0.062 |

In chlorination the time of persistence of the residual active chlorine is of great interest. It is known that in ammoniated water chlorine remains longer than in a water not treated with ammonia. This was shown by our studies (13) and by those of other authors. In order to find out how long residual chlorine persists when different methods are applied for the determination of chlorine dosage, an experiment was made with distilled water to which 0.5 cc. broth was added; the dose of chlorine titrated with 1 mg. $(NH_4)_2SO_4$ and acid at back-titration was equal to 0.246 mg. The corresponding dose found without acid amounted to 0.326 mg.

Upon addition of the above mentioned doses of chlorine the residual chlorine was determined after 1, 2, 4, 6, and 24 hours. Obviously the dose of chlorine titrated without acid was larger than the dose titrated with acid, the residual in the first keeps much longer than in the last.

These are only a few of a large number of experiments carried out during our study. The principal theses of our method have received an experimental confirmation. Before recommending our findings for practical use we believe it necessary first to verify them under plant conditions.

## CONCLUSIONS

1. Determination of chlorine dose according to the chlorine demand, found by means of the now existing iodometric method, is unsatisfactory for treatment of water with ammonia-chlorine.

This method does not take into consideration the influence of the initial dosage of chlorine on the chlorine demand and the influence of ammonia on the same. The value of chlorine demand, as found according to this method, may prove considerably greater than the dose that is really required, and in some instances the dose of chlorine may be found much smaller than is wanted.

2. The methods now in use for determining the chlorine demand likewise do not consider the influence of acidification during back-titration of the residual chlorine.

The chlorine demand determined with the iodometric method without addition of acid during back-titration is greater than the value determined in the presence of acid. The value of chlorine demand is larger in the first instance by the amount of half-bound chlorine. This is liberated on addition of acid and the amount of absorbed chlorine is thus diminished.

3. The amount of half-bound chlorine depends on the chemicals contained in the water. The formation of half-bound chlorine depends on the presence of certain kinds of organic matter (in our experiments it was meat-peptone broth), as well as on the pH value of the water. When the amount of these substances and the pH value increase, the half-bound chlorine is likewise increased. Raising the initial dose of chlorine also increases the half-bound chlorine. Half-bound chlorine may attain considerable values and in ammoniated water may even surpass the amount of absorbed chlorine.

4. The dose of chlorine titrated in ammoniated water with addition of acid is lower (sometimes amounting to a fraction of the dose titrated without acid). This dose does not always produce the necessary bactericidal effect and is found insufficient, especially when the pH of the water is high.

5. The iodometric method which we propose for determining the

dose of chlorine for ammoniated water takes into consideration the value of initial chlorine, the value of half-bound chlorine and the diminished chlorine demand of water upon influence of ammonia. When so determined the chlorine dose always produces a fair bactericidal effect. When the dose of chlorine is determined according to our method the residual persists for a sufficiently long period.

6. Further verification of this method in plant practice is necessary to demonstrate its practical importance.

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## NOTE ON THE DETERMINATION OF CHLORINE*

By T. E. Larson

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Haase and Gad (1), described a method for the determination of free and half-free chlorine by means of di-methyl-*p*-phenylendiamin in phosphoric acid, in which nitrites up to 5.0 p.p.m. and iron up to 1.0 p.p.m. do not interfere. Since phosphoric acid is known to repress ferric ion these authors results are due in part at least to the change in acid. With this in mind phosphoric acid was used instead of hydrochloric for preparation of the *o*-tolidine reagent for a comparative series of tests. The following is a summary of the results of tests with this reagent and that of Haase and Gad.

A. *O*-tolidine was very soluble in phosphoric acid. The color produced was equal to that produced by using the Standard Methods (2) reagent, and more stable. Iron did not interfere when as high as 3 p.p.m. were present if 30 or 50 per cent $H_3PO_4$ reagent was used, but if 10 per cent $H_3PO_4$ reagent was used, 1 p.p.m. iron was the limit before interference was noticeable. Nitrites (a) gave immediate color with Standard Method (10 per cent HCl) reagent, (b) gave immediate but slight color with 30 per cent HCl (Standard Methods modified) reagent, (c) gave color after 10 minutes with 30 or 50 per cent $H_3PO_4$ reagent, and (d) gave immediate color with (15 per cent HCl + 15 per cent $H_3PO_4$) reagent. Chloramine gave relatively rapid color formation if 30 per cent HCl reagent was used but required 30 minutes to 1 hour to develop if 10 per cent HCl, or 10 per cent, 30 per cent, or 50 per cent $H_3PO_4$ reagent was used.

It was concluded that the use of $H_3PO_4$ rather than HCl in preparation of the *o*-tolidine was advantageous only in case iron was the only interfering substance, and that the greater the amount of iron present the greater the amount of $H_3PO_4$ was required. Since free $Cl_2$ and nitrites do not exist in the same solution and since the time for color development by $NH_2Cl$ was even longer than that for nitrites with the phosphate reagent, 30 per cent HCl (Standard

* Contributed record of research.

Methods modified) reagent was definitely better than the $H_3PO_4$ reagent. Also the nitrite color when produced by $H_3PO_4$ reagent was stronger than that produced by the 30 per cent HCl reagent.

B. The *p*-aminodimethylaniline reagent of Haase and Gad was prepared, and a few tests showed that the color by chlorine was immediate but faded rapidly if present in concentrations higher than .7 p.p.m.; color by chloramine was very slow (15–30 minutes) to develop; color by iron was not evident if less than 1 p.p.m. was present; and color by nitrites was momentary, only in the higher concentrations.

It was concluded that, since the color produced by $Cl_2$ or $NH_2Cl$ was so unstable, the reagent was not more advantageous than the 30 per cent *o*-tolidine reagent. This agrees with conclusions from earlier studies made in this laboratory (3).

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# GOLD CHLORIDE PERMANENT STANDARDS FOR RESIDUAL CHLORINE*

By R. D. Scott

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When permanent standards are used in colorimetric determinations it is imperative that, if accuracy is to be attained, their colors closely simulate those developing in the actual determinations.

In determining residual chlorine the potassium dichromate-copper sulphate standards of Standard Methods of Water Analysis provide a satisfactory match for the ortho-tolidine-chlorine color only in 300 m.m. depth of liquid. For other depths the color is not simulated.

At the 1935 Ohio Conference on Water Purification the writer presented a system of color standards prepared from buffered chromate-dichromate solutions which match the ortho-tolidine-chlorine color for any depth of liquid (1) (2). These standards have, however a disadvantage in that on standing for as long as one month a slight precipitate is deposited, apparently due to some impurities in the chemicals available.

Accordingly the study was continued. It was finally observed that gold chloride in hydrochloric acid solution has a color which matches, apparently exactly in hue and intensity that produced by ortho-tolidine and chlorine. The standards prepared do not deposit any sediment and are reasonably permanent as no fading has been noted after three months standing in diffuse daylight. Accordingly these standards, as is the case with the chromate-dichromate standards may be used in 100 m.l., 50 m.l. or any other definite volume and the sample may be matched with standard either through length or breadth of tube.

Gold chloride is supplied in sealed glass ampoules containing 15 grains or slightly less than one gram. Assay of several ampoules has indicated the gold content to be quite uniform, not deviating greatly from 0.47 gram of gold per ampoule. (Gold chloride, $AuCl_3 \cdot 3H_2O$,

* Contributed record of research. Read at the Ohio Water Purification Conference, Cincinnati, October, 1937.

15 grains, 0.9721 gram theoretically contains 0.536 gram of gold.) One ampoule, at present costing $1.24, is sufficient for the preparation of a series of color standards up to and including 0.5 p.p.m. chlorine. The brand and grade we have used is Merck's Reagent.

The standards are prepared as follows: The ampoule is thoroughly cleaned with pumice, first scraping off the label, rinsed and dried. It is placed in a glass-stoppered bottle and broken. One hundred cubic centimeters of distilled water is added and the bottle is shaken until the gold chloride is completely dissolved.

Volumes as shown in table 1 are then made up to 100 cc. with 1 in 5 hydrochloric acid (400 m.l. concentrated HCl solution, specific gravity 1.19 diluted to 2000 m.l.) in Nessler tubes of any uniform diameter. The tubes of standards are closed when not in use, with

TABLE 1

*Permanent chlorine standards*

| CHLORINE | GOLD CHLORIDE SOLUTION | CHLORINE | GOLD CHLORIDE SOLUTION |
|---|---|---|---|
| *p.p.m.* | *millilitres* | *p.p.m.* | *millilitres* |
| 0.01 | 0.15 | 0.20 | 5.5 |
| 0.02 | 0.38 | 0.25 | 7.3 |
| 0.03 | 0.55 | 0.30 | 9.1 |
| 0.05 | 1.00 | 0.40 | 13.2 |
| 0.07 | 1.55 | 0.50 | 18.0 |
| 0.10 | 2.35 | 0.70 | 26.0 |
| 0.15 | 3.8 | 1.00 | 39.0 |

stoppers which are not allowed to come in contact with the solution in order to prevent any reduction of gold chloride. The chlorine concentrations used in adjusting the standards were prepared from calcium hypochlorite solution containing 20 to 25 p.p.m. Cl, as determined closely by iodometric titrations twice daily. From this stock solution dilutions were freshly prepared using distilled water previously treated with 1 p.p.m. Cl, then boiled until free from chlorine and cooled.

If desired the exact strength of the stock gold chloride solution may be determined. To 10 ml. add 25 ml. of N/20 sodium oxalate. Place on a water bath for an hour or until the gold has precipitated completely and the supernatant liquid is clear and colorless. Add 1 ml. of 1-3 sulphuric acid and titrate the excess oxalate with N/40 $KMnO_4$.

Each ml. of N/20 oxalate solution required is equivalent to 3.287 milligrams of gold (Au.). As a check, filter off the flocculent gold precipitate on a small ashless filter paper, wash dry, ignite and weigh as metallic gold. If the gold content of 10 ml. of the stock solution differs appreciably from 0.0470 gram, take proportionate volumes for the standards.

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# THE DYNAMICS OF SEDIMENTATION*

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## I. Introduction

The object of this paper is to formulate the general problem of the settling of solids through a turbulent viscous fluid. A precise definition of turbulence is first given, and in terms of this a consistent dynamical theory of sedimentation is developed. This is a basic investigation. No attempt has been made here to apply the results to the design of settling tanks; such attempts must wait for the experimental determination of the relation between discharge velocities of tanks and the turbulent velocity as introduced in this paper.

In order to make this paper accessible to a wide circle of readers it has been written as descriptively as the nature of the subject and the aims of this investigation permit. The more recondite arguments have been relegated to an appendix. Even here a great deal of mathematical detail has been omitted, but the experienced mathematical reader will find nothing essential lacking, and sufficient references have been given to enable the less experienced to fill in the gaps with little trouble.

## II. The Sediment

In a paper entitled *Sedimentation in Quiescent and Turbulent Basins* (1) the writer has shown explicitly that the problem of sedimentation consists of two essentially distinct parts: the first is the analysis of the hydraulic behavior of the suspended solids, the second is the analysis of the dynamical properties of the basin. In the Proceedings paper (1) an exhaustive investigation of the first part of the problem has been given. It is there shown how from a test sample of the suspended material the hydraulic values of its constituent parts may be determined (the hydraulic value of a particle, or of a

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globule of particles, being defined as the speed with which it settles in a still basin). If there is a quantity B of material suspended in a fluid such that a part $B_1$ settles out at the rate of $v_1$ ft./min., a part $B_2$ at the rate of $v_2$ ft./min., and so on, then the set of ratios $b_1 = B_1/B$, $b_2 = B_2/B$, $b_3 = B_3/B$, .... is called the distribution of hydraulic values. Figure 1 shows two types of distribution. In (*a*) is shown the distribution of a material that consists of a mixture of six distinct hydraulic types. It is important to notice that the material that makes up one of these parts, $b_2$ for instance, need not be, and usually is not, homogeneous in size or chemical or physical constitution. All the distribution says is that there is a fraction $b_2$ of the material that falls with a velocity $v_2$ ft./min. In (*b*) is shown a case where the distribution varies continuously over a range of hydraulic values, but in this case any convenient interval in $v$ may be taken and the $b$'s computed for the corresponding average $v$'s.

In this paper it will be assumed that the distribution of hydraulic values may be found and attention will be fixed on that portion of the suspended solids, $B$, whose hydraulic value is $v$. It is obvious that the complete process will simply be a sum of its separate parts.

## III. THE BASIN

In the Proceedings paper the writer left the second half of the problem unsolved except in a special limiting case which he called critical turbulence. The solution obtained in this case made use of the assumptions adopted by the late Allen Hazen (2). The present investigation proceeds from a radically different point of view. The general formulation of the problem is based only on the fundamental principle of mechanics that a particle of matter cannot change its velocity in a given direction unless it receives an impulse in that direction.

When a particle settles through a viscous liquid, it will attain a constant speed as soon as the force of gravity is exactly balanced by the upward viscous resistance. This happens very quickly in a fluid as viscous as water. If this speed ever changes, it is because in its journey downward the particle passes through a region of the fluid which has a vertical component of velocity. At all times (except during negligible intervals of acceleration) the downward speed of the particle is the algebraic sum of its hydraulic value and the vertical velocity component of the fluid. In figure 2 is shown the path of a particle falling through a stream that has only hori-

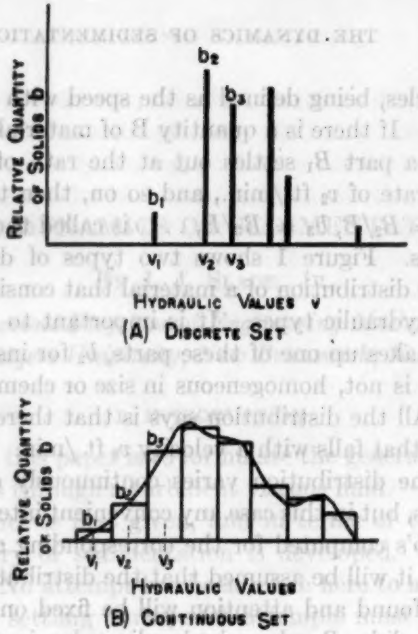


FIG. 1 DISTRIBUTION OF HYDRAULIC VALUES

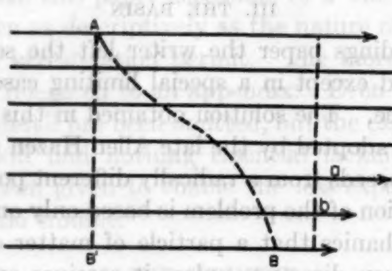


FIG. 2 PATH OF PARTICLE THROUGH PARALLEL FLOW

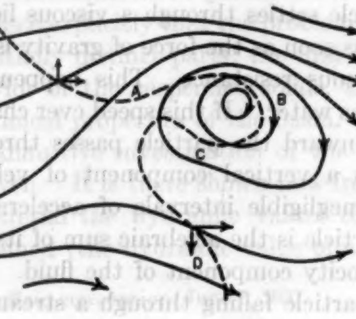


FIG. 3 PATH OF PARTICLE NEAR VORTEX

zontal velocities. In its downward journey the particle receives no vertical impulses so that the time it takes to reach $B$ is the same as that in which it would have reached $B'$ had the water been still. Except for a longitudinal spreading of the sediment at the bottom sedimentation through strictly parallel horizontal flow is identical with quiescent sedimentation.

In any viscous fluid like water, however, the frictional forces between layers that have different speeds, such as $a$ and $b$ in figure 2, tend to produce small vortices, so that viscous flow is never stream-lined. Figure 3 shows the neighborhood of a small vortex. In the region $A$ the particle whose path is indicated by the dotted line is receiving upward impulses, and its downward speed is retarded. At $B$ it is being dragged downward with greater speed. At $C$ it is carried upward. At $D$ the impulses are downward again. This particle will fall in a greater or lesser time than its normal time of subsidence in a still basin depending on whether it receives an excess of upward or downward impulses in its journey.

So far as settling is concerned (and disregarding the longitudinal spread of the sediment at the bottom of the tank), this vertical motion is the turbulence. A precise measure of turbulence may be defined in this way: In any state of motion the fluid possesses a definite amount of kinetic energy $E$. This energy may be divided into two parts, $E = E_H + E_T$, where $E_H$ is that part of it due to the horizontal motion and $E_T$ is that due to the vertical motion. This turbulent energy $E_T$ may be taken as the measure of the turbulence, but it is more convenient to adopt another quantity derived from it. If $M$ is the total mass of the fluid, then $E_T = (M/2)v_T^2$, where $v_T$ is an equivalent vertical velocity. This equivalent velocity, which the writer calls the turbulent velocity, is unique and is equal to the root-mean-square of all the vertical velocities in the basin (See Appendix I (a).) This turbulent velocity will fluctuate somewhat from instant to instant, so that, instead of its instantaneous value, its long-time average value will be adopted as the measure of turbulence. A long-time interval would be an interval comparable to the time of retention. A given tank, then, operating under given conditions, will be characterized by a definite turbulent velocity. It is worthy of note that $v_T$ is a real and precisely defined characteristic of a tank regardless of whether it can be determined or not. In what follows it will be assumed that $v_T$ is known; its determination will be discussed below.

Now, $v_T$ taken by itself is not sufficient to determine the performance of a tank. This velocity is a type of average (root-mean-square) used frequently in statistical investigations. Some of the vertical velocities in the tank will be greater than this value, some smaller; some velocities will be positive (downward), some negative (upward). The average value of these velocities must be zero, since the net vertical flow of the basin is zero. The performance of the tank will be known only when the distribution of these velocities is known. This distribution will fluctuate from time to time, but during a long time interval it will adjust itself so as to give the greatest dynamical stability to the flow. In Appendix I (a) it is shown that this distribution is "normal" (for practical purposes, at any rate). That is, it is the same distribution that controls accidental errors, and it has $v_T$ as its standard deviation and zero as its mean. In such a distribution 32 percent of the vertical velocities are greater in absolute value, 68 percent smaller, than $v_T$. Since the normal distribution is completely determined when its mean and standard deviation are given, the dynamical characteristics of a tank, insofar as they affect settling, will be completely determined when $v_T$ is known. This distribution is shown in figure 4.

## IV. TURBULENT SEDIMENTATION

We may now consider a basin of depth $h$ in which there is initially suspended a quantity $B$ of solid material of hydraulic value $v$ with a uniform vertical distribution. If the liquid in the basin is still, then the amount of material remaining in suspension at the end of time $t$ is $B_t = B(1 - vt/h)$ (3). If the basin has a turbulent velocity $v_T$, then, as indicated above, it has vertical velocities $u_1$, $u_2$, $u_3$, .... distributed normally about the value 0 with a standard deviation equal to $v_T$. That part of the sediment which is passing through a region of the fluid that has a vertical component of velocity $u$ will be settling as if it had a hydraulic value $v' = v + u$, and, if the turbulent energy of the fluid is always accessible to the solids, the fraction of the material dropping with velocity $v'$ is the same as the fraction of the fluid that has the velocity $u$. As the process of settling continues, there is a constant interchange of energy throughout the various parts of the basin and intervals of time; no single portion of the sediment will be dropping continuously with a velocity $v'$. But the individuality of the particles is inessential; all that matters is that a definite proportion of the solids, on the average, is falling with velocity $v'$.

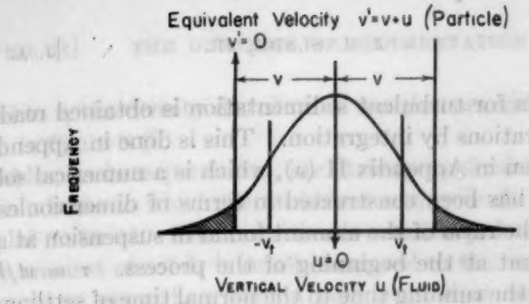


Fig. 4 Distribution of Vertical Velocities

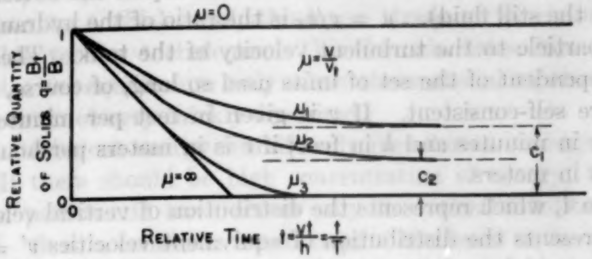


Fig. 5 Turbulent Sedimentation

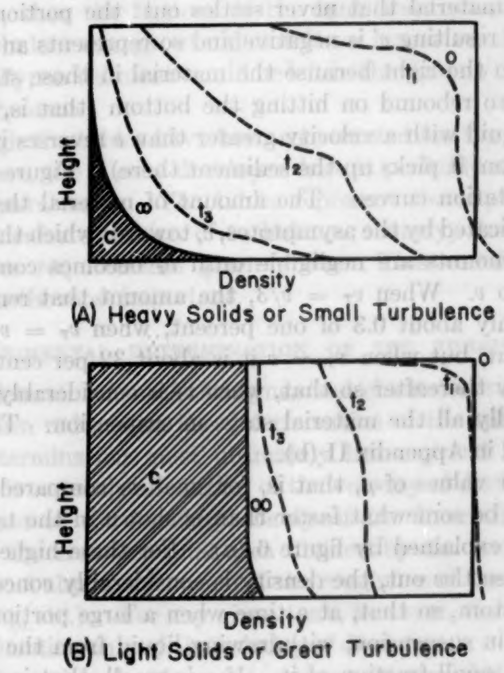


Fig. 6 Vertical Distribution of Density

The formula for turbulent sedimentation is obtained readily from these considerations by integration. This is done in Appendix I (b). The table given in Appendix II (a), which is a numerical solution of this formula, has been constructed in terms of dimensionless ratios. $q = B_t/B$ is the ratio of the amount found in suspension at any time $t$ to the amount at the beginning of the process. $\tau = vt/h = t/T$ is the ratio of the running time to the normal time of settling (that is, the time it takes a particle to fall from the top to the bottom of the basin in the still fluid). $\mu = v/v_T$ is the ratio of the hydraulic value of the particle to the turbulent velocity of the tank. These ratios are independent of the set of units used so long, of course, as these units are self-consistent. If $v$ is given in feet per minute, then $t$ must be in minutes and $h$ in feet; if $v$ is in meters per hour, $t$ is in hours, $h$ in meters.

Figure 4, which represents the distribution of vertical velocities $u$, also represents the distribution of equivalent velocities $v' = v + u$ of the material, as indicated above, where the mean of these is $v$ (since the mean of the $u$'s is zero). The shaded areas give the fraction of the material that never settles out: the portion to the left because the resulting $v'$ is negative and so represents an upward velocity; that to the right because the material in those states has enough energy to rebound on hitting the bottom (that is, when a region of the liquid with a velocity greater than $v$ reverses its direction at the bottom it picks up the sediment there). Figure 5 shows typical sedimentation curves. The amount of material that never settles out is indicated by the asymptotes, $c$, towards which the curves tend. These amounts are negligible until $v_T$ becomes comparable in magnitude to $v$. When $v_T = v/3$, the amount that remains in suspension is only about 0.3 of one percent, when $v_T = v/2$, it is about 5 per cent, but when $v_T = v$ it is about 32 per cent, and it increases rapidly thereafter so that, when $v_T$ is considerably greater than $v$, practically all the material stays in suspension. This relation is tabulated in Appendix II (b).

For the larger values of $\mu$, that is, for small $v_T$ compared with $v$, the process will be somewhat faster than is shown in the tables for the reason best explained by figure 6 ($a$). For these higher ratios, as the material settles out, the density is considerably concentrated towards the bottom, so that, at a time when a large portion of the material is still in suspension, withdrawing liquid from the top will carry off only a small fraction of it. If $\mu$ is small, that is, $v_T$ great

compared to $v$, the density distribution remains practically constant, as shown in figure 6 (*b*).

The tables, as here presented, are strictly applicable only to the process in a basin in which the turbulent energy is always accessible to the solid material. This condition is satisfied provided that:

1. The amount of solids is small compared to the volume of fluid. The formula as given here should hold even close to the point of saturation, except that in this latter case the sediment at the bottom would soon be a thick mud, the lower strata of which would be inaccessible to the turbulence. This formula takes into account final statistical equilibrium only when the material at the bottom is loosely and not very thickly deposited.

2. The turbulent energy is uniformly distributed throughout the basin. If there should be high concentration of energy, the concentrated part would have access only to a small fraction of the material. Such a condition would require the basin to be analyzed in two or more parts.

3. There is no marked stratification of the fluid. Suppose, for instance, that the bottom of the tank were practically stagnant. The turbulent energy would not have access to the bottom and, eventually, no sediment would be left in the fluid no matter how great the turbulence.

Though the tables have been constructed for the case of uniform turbulence, the theory developed here is perfectly applicable to the more general cases. The tables represent the explicit solution for one particular condition of turbulence; namely, that which satisfies the three requirements given above. This condition the writer considers the most important.

## V. EXPERIMENTAL DETERMINATION OF THE TURBULENT VELOCITY

The determination of the dynamical characteristics of a tank has so far been reduced to the determination of the turbulent velocity. This determination must ultimately be empirical. The turbulent velocity is certainly a function of the velocity of discharge and of the dimensions of the basin (probably of the hydraulic mean depth; that is, the cross-sectional area divided by the wetted perimeter), and it must also be a function of the roughness of the walls (condition of baffling, etc.). For any given tank, however, the turbulent velocity is a function only of the velocity of discharge.

The foregoing development suggests a straightforward, though

possibly not very practicable, method of determining this relation. If a material of practically constant hydraulic value is allowed to settle in a tank, a definite amount of it will remain in permanent suspension after the process has gone on a long interval of time, and this amount has a definite relation to the turbulent velocity, as shown in Appendix II (b). Suppose the material has a hydraulic value of 1.5 ft./hr. and that 10 percent of it is found to remain in permanent suspension. For $c = 10$ percent the table gives $\mu = 1.654$. That is, $\mu = v/v_T = 1.5/v_T = 1.654$, and $v_T = 0.97$ ft./hr. (At the end of the process, especially where $\mu$ is large, the density distribution of the suspended solids is variable vertically, so that samples would have to be taken from several levels.) This could be done for several rates of discharge to obtain the relation between the turbulent velocity and rate of discharge.

Another, and perhaps more practicable method, would be to calibrate a ten or twelve gallon cylinder with a rotor for turbulent velocity against angular speed of the rotor. This could be done in the same manner as outlined above. Then for any material the speed of the rotor could be regulated until the relative sedimentation curve for the cylinder (that is, the curve in the dimensionless $\mu$ and $\tau$ ratios) coincided with that for the tank. From the calibration curve for the cylinder the turbulent velocity of the tank would be determined.

## VI. CONCLUSIONS

The general formulation of the problem presented here is based on well-established dynamical principles. No arbitrary elements or assumptions enter the discussion until the form of the vertical velocity distribution is introduced. The arguments offered in Appendix I (a) in support of the normal distribution seem valid, and the writer adopts it with confidence. However, should the dynamical constraints of the problem affect certain frequencies more than the writer believes they do, then a more suitable distribution function must be substituted for the normal. The rest of the analysis would remain unaltered. This deductive flaw may be removed by determining the distribution experimentally by the method outlined in Appendix I (c). The precision with which this may be done is a matter of experimental technique. The problem of turbulent sedimentation is thus brought up to a high order of determinateness.

The final formula (Formula (7), Appendix I (b)), from which the

tables have been computed, or one obtained from another distribution function, should be regarded as a statement of the probability that a certain amount of sediment be found in suspension after the lapse of time $t$. The nature of the problem does not admit a more precise formulation. This formula gives the average performance of the tank; if the tank operates quite uniformly, the fluctuations from the average will never be great.

The maturing influence of long reflection since the writer first attempted the solution of this problem has not diminished his conviction, expressed before, that the design of settling tanks may be placed on as precise a foundation as that, for instance, on which rests the design of trusses.

## APPENDIX I

(*a*) *Distribution of Velocities.* Divide the tank into a large number, $n$, of small cells of equal volume. Let $u_1, u_2, u_3, \ldots.$ represent all possible values of the vertical components of the velocity of the fluid through these cells. Now, the *a priori* probability of a velocity $u$ is proportional to the frequency of this velocity. Let $p_s$ be the relative frequency associated with the velocity $u_s$, $s = 1, 2, 3, \ldots.$ If we take an arbitrary set of $n$ $u$'s, the probability that this set be found in the tank is $\pi p_s$, where this symbol stands for the product $p_1 p_2 \ldots. p_n$ (4). Also at any time $\Sigma u_s = 0$, since the net vertical flow is null. (It may happen that the vertical flow is not null, as would be the case if the water were introduced and withdrawn at different levels. In that case the analysis would be modified only in having the distribution about some average value $\bar{u}$ instead of the zero value.) Over a long-time average $\pi p_s$ must be the maximum obtainable from all possible combinations.

That is, we must have

(*1*) $\pi p_s$ a conditioned maximum,

(*2*) $\Sigma u_s = 0$.

If there were no conditions imposed on (1), then the relations (*1*) and (*2*) taken simultaneously would lead uniquely to the normal distribution (5). The condition that must be imposed consists in requiring the distribution to be such as to lead to maximum dynamical stability compatible with the constraints of the flow. Maximum dynamical stability for one unconstrained degree of freedom is obtained when the velocities are normally distributed (6). Further-

more, the tendency towards a normal distribution is very strong. In confined viscous flow there are three principal constraints. The velocities of adjacent cells are not independent; the viscosity affects the greater velocities more than the lesser; there is always a certain amount of directed vertical flow in a tank. The first constraint leads to possible high instantaneous variations, but these would average out in the long run. The second constraint certainly requires that the higher velocities be less frequent than in the normal variation. This will probably not lead to a serious modification since the tank velocities are all small and, consequently, not greatly damped, and in the normal distribution practically all the velocities are less than three times the standard deviation. The third constraint would give certain special velocities higher frequencies than expected.

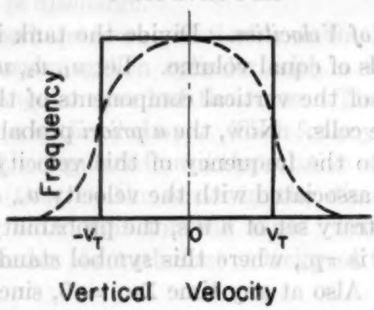


FIG. 7 UNSTABLE DISTRIBUTION OF VELOCITIES

Altogether, the frequency distribution would tend strongly towards the normal, being possibly slightly flatter and somewhat skewed. Now, in this investigation we are interested principally in the region of mediocrity, since the exceptional values have but a negligible effect on the process—being infrequent, they would affect only a small portion of the solids. In this region any distribution that tends to the normal may be replaced by a normal distribution, the actual differences being of no practical consequence.

An artificial distribution is shown in figure 7. Such a distribution would be unstable and could not be maintained for more than a small interval of time even by means of elaborate baffling. Here and there velocities would annul each other, here and there they would be reinforced, so that in the long run the distribution would take the

shape of the dotted line. It may be noted here that Hazen's solution of the problem (7), which the writer modified in the Proceedings paper to obtain the condition he there calls critical turbulence, is possible only under a very special velocity distribution which would be highly unstable. No reasonable distribution leads to uniform vertical density, and in those conditions of high turbulence where the density tends to uniformity, the pick-up from the floor of the basin is considerable.

From these considerations the writer concludes that, for practical purposes (and probably to a much higher degree of approximation than is required for practical purposes), the distribution of vertical velocities is normal with the zero value for a mean. If we let $m$ be the mass of the water in a cell, then the turbulent energy is $E_T = \Sigma(m/2)u_s^2 = (M/2)v_T^2$, and, since $\Sigma m = nm = M$, therefore

$$v_T = \sqrt{\frac{\Sigma u_s^2}{n}} \tag{3}$$

That is, $v_T$ is the standard deviation of the distribution. We may therefore write

$$p(u) = \frac{1}{\sqrt{2\pi}\, v_T} e^{-\frac{u^2}{2v_T^2}} du = \Psi\left(\frac{u}{v_T}\right) d\left(\frac{u}{v_T}\right) = \Psi(x)\, dx \tag{4}$$

(*b*) *The Sedimentation Formula.* In still water the fraction of the solids remaining in suspension at the end of time $t$ is $q = (1 - vt/h)$, where $q = B_t/B$. In a turbulent fluid there is a fraction $p(u)$ dropping with velocity $v' = v + u$, so that of this, over a long-time interval, $p(u)\ (1 - (v + u)t/h)$ remains in suspension provided that $-v \leq u \leq v$. The particles suffering impulses outside this range have too much energy to settle out. The complete process will be the sum of partial processes of this type. Adopting the form of $p(u)$ given by Equation (4), we have

$$q = \int_{-\infty}^{-v} \Psi\left(\frac{u}{v_T}\right) \frac{du}{v_T} + \int_{v}^{\infty} \Psi\left(\frac{u}{v_T}\right) \frac{du}{v_T} + \int_{-v}^{f(t)} \Psi\left(\frac{u}{v_T}\right)\left(1 - \frac{(v+u)t}{h}\right) \frac{du}{v_T} \tag{5}$$

in which the upper limit of integration in the last term depends on how long the process has been going on. At first all particles are falling whose equivalent velocities are in the range $0 < v' \leq 2v$, so

that the upper limit is $v$, but, when $t$ is such that $1 - (v + u)t/h = 0$, the material whose equivalent velocities $v' = v + u > h/t$ will have dropped out (8). Consequently, for $t \geq h/2v$ the upper limit for $u$ is $h/t - v$. This completely determines $f(t)$:

$$(6) \qquad f(t) = \begin{cases} v \text{ for values of } t \leq h/2v \\ h/t - v \text{ for values of } t > h/2v. \end{cases}$$

Letting $x = u/v_T$, $\mu = v/v_T$ $\tau = vt/h$, and

$$P(y) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{y} e^{\frac{-x^2}{2}}\, dx \quad (9),$$

we may write

$$(7) \quad q = 2\int_{\mu}^{\infty} \Psi(x)\, dx + \left(1 - \frac{vt}{h}\right) \int_{-\mu}^{\frac{1}{v_T}f(t)} \Psi(x)\, dx - \frac{v_T t}{h} \int_{-\mu}^{\frac{1}{v_T}f(t)} x\Psi(x)\, dx$$

$$= \begin{cases} 2(1 - P(\mu)) + (1 - \tau)(2P(\mu) - 1) & \text{for } \tau \leq 0.5 \\ (1 + \tau)(1 - P(\mu)) + (1 - \tau)\, P\left[\frac{\mu}{\tau}(1 - \tau)\right] \\ \qquad + \frac{\tau}{\mu}\left(\Psi\left[\frac{\mu}{\tau}(1 - \tau)\right] - \Psi(\mu)\right) & \text{for } \tau > 0.5 \end{cases}$$

The amount of sediment that remains permanently in suspension is

$$(8) \qquad c = q(t \to \infty) = 2(1 - P(\mu)).$$

The tables (a) and (b) of Appendix II are tabulations of equations (7) and (8) respectively.

When the sediment does not have a constant hydraulic value but has a distribution $b_1, b_2, b_3, \ldots.$ of hydraulic values $v_1, v_2, v_3, \ldots.$ then, if $q_1, q_2, q_3, \ldots.$ are the functions obtained from equation (7) for the corresponding parameters $\mu = \mu_1, \mu_2, \mu_3, \ldots.$, the sedimentation formula becomes

$$(9) \qquad q = \Sigma b_s q_s.$$

(c) *Experimental Determination of the Distribution.* If we are able to express the distribution function explicitly in terms of $q$, the rela-

tive quantity of solids in suspension at any time, we may determine the form of the distribution of vertical velocities experimentally, since $q$ is an observable quantity in any case. Now, for any form of $\Psi\left(\frac{u}{v_T}\right)$, it is evident that

$$q = c + \int_{-v}^{g(t)} \Psi\left(\frac{u}{v_T}\right)\left(1 - \frac{u+v}{h}t\right)\frac{du}{v_T} \tag{10}$$

in which $c$ is the relative quantity of solids in permanent suspension, and $g(t)$ is constant for all values of $t$ less than some $t_0$, and equal to $h/t - v$ for $t > t_0$. In particular, $t_0$ might possibly be zero. Differentiating twice with respect to $t$ (10), we get

$$q'' = \begin{cases} 0, & t < t_0 \\ \dfrac{h}{v_T t^3}\Psi\left(\dfrac{h}{v_T t} - \dfrac{v}{v_T}\right), & t > t_0 \end{cases} \tag{11}$$

This may be written

$$\frac{\tau^3}{v}\frac{d^2q(\tau)}{d\tau^2} = \frac{1}{v_T}\Psi\left(\frac{\mu}{\tau} - \mu\right) = \frac{1}{v_T}\Psi\left(\frac{u}{v_T}\right), \quad \tau > \tau_0 \tag{12}$$

where the argument of the distribution function is obtained by means of the transformation $u/v_T = \frac{\mu}{\tau} - \mu$.

The operations on the left side of equation (12) may be performed graphically on the observed $q$. The accuracy of this determination will depend on the accuracy with which $q$ is observed.

| $\tau$ | ∞ | 10μ | 9μ | 8μ | 7μ | 6μ | 5μ | 4μ | 3μ | 2.8μ | 2.6μ | 2.4μ | 2.2μ | 2.0μ | 1.9μ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 |
| .1 | .9000 | .9000 | .9000 | .9000 | .9000 | .9000 | .9000 | .9000 | .9001 | .9006 | .9009 | .9016 | .9028 | .9045 | .9057 |
| .2 | .8000 | .8000 | .8000 | .8000 | .8000 | .8000 | .8000 | .8000 | .8004 | .8012 | .8018 | .8033 | .8055 | .8091 | .8115 |
| .3 | .7000 | .7000 | .7000 | .7000 | .7000 | .7000 | .7000 | .7000 | .7007 | .7017 | .7028 | .7049 | .7083 | .7136 | .7172 |
| .4 | .6000 | .6000 | .6000 | .6000 | .6000 | .6000 | .6000 | .6000 | .6009 | .6022 | .6037 | .6065 | .6111 | .6177 | .6230 |
| .5 | .5000 | .5000 | .5000 | .5000 | .5000 | .5000 | .5000 | .5000 | .5013 | .5027 | .5046 | .5082 | .5139 | .5227 | .5287 |
| .6 | .4000 | .4000 | .4000 | .4000 | .4000 | .4000 | .4000 | .4002 | .4030 | .4051 | .4083 | .4133 | .4212 | .4329 | .4406 |
| .7 | .3000 | .3000 | .3000 | .3000 | .3000 | .3002 | .3009 | .3031 | .3122 | .3163 | .3222 | .3304 | .3419 | .3578 | .3679 |
| .8 | .2000 | .2002 | .2004 | .2009 | .2019 | .2039 | .2081 | .2167 | .2362 | .2434 | .2520 | .2635 | .2786 | .2985 | .3107 |
| .9 | .1000 | .1059 | .1083 | .1115 | .1162 | .1227 | .1326 | .1485 | .1768 | .1869 | .1980 | .2122 | .2282 | .2529 | .2667 |
| 1.0 | .0000 | .0399 | .0443 | .0499 | .0570 | .0665 | .0798 | .0998 | .1342 | .1450 | .1575 | .1733 | .1930 | .2180 | .2329 |
| 1.1 | | .0109 | .0142 | .0188 | .0250 | .0338 | .0467 | .0669 | .1029 | .1143 | .1274 | .1442 | .1649 | .1884 | .2067 |
| 1.2 | | .0024 | .0039 | .0064 | .0103 | .0167 | .0272 | .0454 | .0832 | .0917 | .1051 | .1221 | .1433 | .1702 | .1862 |
| 1.3 | | .0005 | .0010 | .0021 | .0042 | .0082 | .0160 | .0313 | .0639 | .0750 | .0883 | .1049 | .1265 | .1537 | .1696 |
| 1.4 | | .0001 | .0002 | .0013 | .0017 | .0041 | .0097 | .0221 | .0518 | .0625 | .0754 | .0920 | .1132 | .1405 | .1566 |
| 1.5 | | .0000 | .0001 | .0002 | .0007 | .0021 | .0060 | .0159 | .0428 | .0531 | .0654 | .0816 | .1076 | .1297 | .1461 |
| 1.6 | | | .0000 | .0001 | .0003 | .0012 | .0046 | .0118 | .0336 | .0456 | .0575 | .0734 | .0940 | .1209 | .1372 |
| 1.7 | | | | .0000 | .0002 | .0006 | .0025 | .0089 | .0307 | .0399 | .0512 | .0666 | .0850 | .1134 | .1296 |
| 1.8 | | | | | .0001 | .0004 | .0017 | .0068 | .0267 | .0353 | .0462 | .0611 | .0810 | .1074 | .1235 |
| 1.9 | | | | | .0000 | .0002 | .0012 | .0054 | .0238 | .0316 | .0420 | .0565 | .0761 | .1021 | .1181 |
| 2.0 | | | | | | .0002 | .0008 | .0043 | .0206 | .0285 | .0386 | .0527 | .0718 | .0976 | .1135 |
| 2.2 | | | | | | .0001 | .0005 | .0028 | .0167 | .0239 | .0332 | .0467 | .0651 | .0902 | .1059 |
| 2.4 | | | | | | .0001 | .0003 | .0020 | .0140 | .0207 | .0285 | .0422 | .0600 | .0846 | .1000 |
| 2.6 | | | | | | .0001 | .0002 | .0014 | .0120 | .0183 | .0264 | .0387 | .0561 | .0802 | .0953 |
| 2.8 | | | | | | .0001 | .0001 | .0011 | .0112 | .0165 | .0242 | .0357 | .0529 | .0766 | .0918 |
| 3.0 | | | | | | .0001 | .0001 | .0007 | .0095 | .0151 | .0226 | .0339 | .0503 | .0736 | .0878 |
| 3.5 | | | | | | .0000 | .0001 | .0004 | .0078 | .0127 | .0193 | .0300 | .0462 | .0681 | .0825 |
| 4.0 | | | | | | | .0001 | .0004 | .0065 | .0118 | .0173 | .0275 | .0425 | .0644 | .0785 |
| 4.5 | | | | | | | .0001 | .0003 | .0058 | .0103 | .0160 | .0257 | .0402 | .0616 | .0756 |
| 5.0 | | | | | | | .0001 | .0002 | .0052 | .0096 | .0150 | .0244 | .0386 | .0596 | .0733 |
| 6.0 | | | | | | | .0001 | .0002 | .0046 | .0087 | .0137 | .0226 | .0363 | .0569 | .0702 |
| 7.0 | | | | | | | .0001 | .0002 | .0042 | .0083 | .0128 | .0215 | .0351 | .0548 | .0680 |
| 8.0 | | | | | | | .0001 | .0001 | .0039 | .0080 | .0123 | .0207 | .0337 | .0535 | .0665 |
| 9.0 | | | | | | | .0001 | .0001 | .0037 | .0078 | .0118 | .0201 | .0326 | .0523 | .0654 |
| 10.0 | | | | | | | .0001 | .0001 | .0035 | .0077 | .0116 | .0197 | .0324 | .0517 | .0645 |
| ∞ | | | | | | | .0000 | .0001 | .0027 | .0053 | .0093 | .0164 | .0278 | .0455 | .0574 |

IX II

(A)

| 1.5μ | 1.4μ | 1.3μ | 1.2μ | 1.1μ | 1.0μ | .9μ | .8μ | .7μ | .6μ | .5μ | .4μ | .3μ | .2μ | .1μ | .08μ | .06μ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 | 1.0000 |
| .9134 | .9161 | .9194 | .9230 | .9217 | .9317 | .9368 | .9424 | .9484 | .9548 | .9617 | .9689 | .9764 | .9841 | .9920 | .9936 | .9952 |
| .8267 | .8323 | .8387 | .8460 | .8543 | .8635 | .8736 | .8847 | .8968 | .9097 | .9234 | .9378 | .9528 | .9683 | .9841 | .9872 | .9904 |
| .7401 | .7485 | .7581 | .7690 | .7814 | .7952 | .8104 | .8271 | .8452 | .8645 | .8851 | .9067 | .9292 | .9524 | .9761 | .9809 | .9856 |
| .6534 | .6646 | .6774 | .6921 | .7085 | .7269 | .7472 | .7695 | .7936 | .8194 | .8468 | .8757 | .9057 | .9366 | .9681 | .9745 | 9809 |
| .5668 | .5808 | .5968 | .6151 | .6358 | .6587 | .6841 | .7119 | .7420 | .7742 | .8085 | .8446 | .8821 | .9207 | .9602 | .9674 | .9761 |
| .4884 | .5055 | .5250 | .5471 | .5719 | .6001 | .6296 | .6625 | .6980 | .7360 | .7763 | .8194 | .8599 | .9075 | .9535 | .9628 | .9721 |
| .4265 | .4468 | .4696 | .4952 | .5236 | .5549 | .5875 | .6261 | .6658 | .7082 | .7529 | .7997 | .8453 | .8980 | .9488 | .9590 | .9692 |
| .3787 | .4016 | .4271 | .4555 | .4868 | .5211 | .5583 | .5986 | .6416 | .6872 | .7354 | .7856 | .8376 | .8909 | .9452 | .9562 | .9671 |
| .3418 | .3662 | .3943 | .4256 | .4581 | .4950 | .5313 | .5772 | .6230 | .6710 | .7207 | .7746 | .8293 | .8854 | .9423 | .9540 | .9653 |
| .3132 | .3395 | .3687 | .4008 | .4360 | .4743 | .5157 | .5603 | .6078 | .6580 | .7108 | .7658 | .8229 | .8810 | .9402 | .9521 | .9641 |
| .2907 | .3181 | .3482 | .3815 | .4180 | .4577 | .5005 | .5466 | .5956 | .6475 | .7020 | .7587 | .8173 | .8775 | .9384 | .9507 | .9629 |
| .2724 | .3006 | .3316 | .3650 | .4034 | .4439 | .4880 | .5353 | .5856 | .6381 | .6946 | .7528 | .8128 | .8743 | .9369 | .9494 | .9621 |
| .2577 | .2863 | .3180 | .3529 | .3912 | .4377 | .4776 | .5258 | .5772 | .6320 | .6887 | .7477 | .8090 | .8718 | .9353 | .9485 | .9612 |
| .2455 | .2745 | .3067 | .3421 | .3810 | .4232 | .4688 | .5178 | .5700 | .6251 | .6831 | .7434 | .8058 | .8696 | .9346 | .9476 | .9606 |
| .2353 | .2646 | .2971 | .3329 | .3723 | .4151 | .4613 | .5116 | .5638 | .6198 | .6781 | .7397 | .8026 | .8677 | .9335 | .9468 | .9601 |
| .2267 | .2563 | .2884 | .3252 | .3648 | .4080 | .4548 | .5050 | .5585 | .6151 | .6746 | .7365 | .8005 | .8660 | .9327 | .9461 | .9595 |
| .2193 | .2490 | .2817 | .3184 | .3584 | .4019 | .4491 | .4998 | .5538 | .6109 | .6711 | .7336 | .7985 | .8646 | .9320 | .9455 | .9591 |
| .2130 | .2428 | .2762 | .3125 | .3527 | .3966 | .4441 | .4952 | .5497 | .6084 | .6680 | .7301 | .7964 | .8633 | .9313 | .9450 | .9588 |
| .2075 | .2373 | .2705 | .3073 | .3477 | .3918 | .4396 | .4905 | .5460 | .6041 | .6649 | .7289 | .7947 | .8622 | .9307 | .9445 | .9584 |
| .2021 | .2325 | .2658 | .3027 | .3433 | .3876 | .4357 | .4874 | .5427 | .6012 | .6627 | .7269 | .7932 | .8611 | .9303 | .9442 | .9581 |
| .1946 | .2245 | .2557 | .2949 | .3358 | .3805 | .4289 | .4812 | .5370 | .5962 | .6584 | .7234 | .7905 | .8594 | .9293 | .9434 | .9575 |
| .1881 | .2197 | .2514 | .2886 | .3297 | .3746 | .4234 | .4761 | .5323 | .5921 | .6549 | .7205 | .7883 | .8578 | .9286 | .9427 | .9570 |
| .1829 | .2127 | .2462 | .2834 | .3246 | .3697 | .4177 | .4717 | .5284 | .5886 | .6519 | .7180 | .7864 | .8565 | .9279 | .9421 | .9566 |
| .1785 | .2083 | .2417 | .2790 | .3203 | .3656 | .4149 | .4681 | .5251 | .5856 | .6494 | .7159 | .7848 | .8555 | .9273 | .9417 | .9563 |
| .1748 | .2046 | .2380 | .2753 | .3163 | .3617 | .4117 | .4650 | .5222 | .5831 | .6472 | .7141 | .7834 | .8548 | .9268 | .9414 | .9560 |
| .1678 | .1974 | .2308 | .2684 | .3095 | .3551 | .4049 | .4588 | .5165 | .5780 | .6428 | .7104 | .7806 | .8534 | .9259 | .9408 | .9553 |
| .1628 | .1922 | .2255 | .2628 | .3043 | .3501 | .4000 | .4542 | .5123 | .5742 | .6395 | .7078 | .7785 | .8522 | .9251 | .9402 | .9548 |
| .1590 | .1884 | .2214 | .2588 | .3004 | .3497 | .3963 | .4507 | .5090 | .5712 | .6370 | .7051 | .7760 | .8512 | .9246 | .9396 | .9545 |
| .1561 | .1853 | .2185 | .2557 | .2972 | .3431 | .3933 | .4478 | .5065 | .5689 | .6350 | .7032 | .7757 | .8504 | .9243 | .9392 | .9544 |
| .1519 | .1810 | .2139 | .2511 | .2927 | .3381 | .3890 | .4437 | .5025 | .5655 | .6319 | .7015 | .7738 | .8482 | .9239 | .9385 | .9540 |
| .1490 | .1779 | .2108 | .2484 | .2895 | .3354 | .3858 | .4407 | .4998 | .5629 | .6298 | .6997 | .7723 | .8472 | .9234 | .9379 | .9536 |
| .1469 | .1757 | .2085 | .2456 | .2870 | .3331 | .3836 | .4386 | .4978 | .5611 | .6282 | .6982 | .7713 | .8464 | .9229 | .9376 | .9532 |
| .1453 | .1740 | .2068 | .2437 | .2853 | .3313 | .3818 | .4369 | .4962 | .5596 | .6269 | .6971 | .7704 | .8458 | .9226 | .9375 | .9529 |
| .1432 | .1728 | .2053 | .2423 | .2838 | .3299 | .3804 | .4356 | .4949 | .5586 | .6259 | .6963 | .7699 | .8454 | .9223 | .9374 | .9526 |
| .1336 | .1615 | .1936 | .2301 | .2713 | .3173 | .3681 | .4237 | .4839 | .5485 | .6171 | .6892 | .7642 | .8415 | .9202 | .9362 | .9522 |

(8) SLADE, loc. cit., where this phenomenon, though in a different connection, is discussed in greater detail.

(9) PEARSON, KARL. Tables for Statisticians and Biometricians, Table I. The function P is there called (1 + α)/2 and the function α is called ...

(10) SLADE, loc. cit., where a similar deduction is carried out in detail in the case of the distribution of normal times of subsidence.

Discussion by PROF. L. V. CARPENTER.* The multiplicity of factors affecting sedimentation makes it a very difficult problem to analyze

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TABLE (B)

| c | μ | c | μ | c | μ |
|---|---|---|---|---|---|
| .02 | 2.3263 | .26 | 1.1264 | .50 | .6745 |
| .04 | 2.0537 | .28 | 1.0803 | .55 | .5978 |
| .06 | 1.8808 | .30 | 1.0364 | .60 | .5244 |
| .08 | 1.7507 | .32 | .9945 | .65 | .4538 |
| .10 | 1.6449 | .34 | .9542 | .70 | .3853 |
| .12 | 1.5548 | .36 | .9154 | .75 | .3186 |
| .14 | 1.4758 | .38 | .8779 | .80 | .2533 |
| .16 | 1.4051 | .40 | .8416 | .85 | .1891 |
| .18 | 1.3408 | .42 | .8064 | .90 | .1257 |
| .20 | 1.2816 | .44 | .7722 | .95 | .0627 |
| .22 | 1.2265 | .46 | .7388 | | |
| .24 | 1.1750 | .48 | .7063 | | |

## REFERENCES


(1) SLADE, J. J., JR. Sedimentation in Quiescent and Turbulent Basins. Proc. A. S. C. E. 61: 1435 (1935).

For brevity the paper cited above will hereafter be referred to as the Proceedings paper. For reasons that appeared sufficient at the time the writer adopted a notation in the Proceedings paper which he has since abandoned. Where he has there written *a*, he now writes *t*; where he has there written *t*, he now writes *T* or $h/v$. It should be noted also that the method developed there determines the distribution of the times of subsidence. The corresponding hydraulic values may be obtained from the relation, $vT = h$ ($vt = h$ in the former notation).

(2) HAZEN, ALLEN On Sedimentation. Trans. A. S. C. E. 53: 43 (1904).

(3) HAZEN, *loc. cit.*; also SLADE, *loc. cit.*

(4) FRY, THORNTON C. *Probability and its Engineering Uses.* D. Van Nostrand Co., Inc., New York. P. 48, Art. 21. (1928.)

(5) WILSON, E. B. *Advanced Calculus.* Ginn and Co. P. 386 et seq.

(6) WILSON, WILLIAM *Theoretical Physics.* E. P. Dutton & Co., Inc., New York. Vol. I.

The reader will here find an elementary discussion of the equilibrium of mechanical assemblies.

(7) HAZEN, *loc. cit.*

(8) SLADE, *loc. cit.*, where this phenomenon, though in a different connection, is discussed in greater detail.

(9) PEARSON, KARL *Tables for Statisticians and Biometricians,* Table I. The function *P* is there called $(1 + a)/2$ and the function *u* is called *z*.

(10) SLADE, *loc. cit.*, where a similar deduction is carried out in detail in the case of the distribution of normal times of subsidence.


*Discussion by* PROF. L. V. CARPENTER:* The multiplicity of factors affecting sedimentation makes it a very difficult problem to analyze

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mathematically. Some of the common factors are: (1) period of retention; (2) depth of flow; (3) length of flow; (4) particle size; (5) electrical effects; (6) specific gravity of sediment; (7) viscosity of liquid; (8) concentration of particles; (9) type of baffling; (10) coagulation, etc.

The problem before the water purification plant designer divides itself into three phases. First, the basin itself—which comprises the general shape and size, the types of inlet and outlet arrangements, the baffles, and the method of removal of settling material. This at present can be handled best by the construction of models, being careful that dynamical similarity exists. Morrill (1) shows that in order to have dynamical similarity between a model and a prototype "$n$" times as large in each linear dimension, the velocity in the full size structure must be $\sqrt{n}$ times that in the model. The same relation holds for the detention period.

The second phase of interest to the designer is coagulation. The physical characteristics of the flocculent material have great bearing on sedimentation. So the treatment preliminary to sedimentation is very important. This phase is just partially understood but will not be discussed further here.

The third phase, prediction of performance, appears to be more susceptible of analysis than the other two. The late Allen Hazen (2) in 1904 set forth what has become known as "Hazen's Theory of Sedimentation." Practically all text books have copies of this, and while Hazen's paper contained something of value toward the development of a rational theory, it contains so many invalid assumptions that it is of little practical use to sanitary engineers in design.

Professor Slade (3) in a previous paper developed a theory which unfortunately was premised upon the validity of the assumptions made by the late Allen Hazen. This paper dealt mostly with an analysis of the hydraulic behavior of the suspended solids. It is refreshing to note in the present paper the author has not made assumption, but based his mathematical analysis "on the fundamental principle of mechanics that a particle of matter cannot change its velocity in a given direction unless it receives an impulse in that direction." The analysis is clear and concise and offers much of value to the ultimate understanding of the problem. All of the mathematics are contingent on the turbulent energy being uniformly distributed throughout the basin and there being no marked stratification of the fluid.

Uniformly distributed turbulent energy and lack of stratification are things that the designer has been attempting to get for years,

with little success. Stratification due to temperature is difficult to control. Turbulent energy can be made uniform by proper design. Pearl (4) has set up an expression for the theoretical attraction between the mass of a particle of sediment and the sides of a circular container, and has attempted to show that the particle is attracted to the side of the vessel while the mass of the earth causes it to settle downward through the liquid. His experimental work was performed on a spiral baffled tank similar to the one now sold commercially by the Pittsburgh-Des Moines Steel Company. This type of baffling can be designed to give very good distribution of velocities.

Professor Slade has made an excellent contribution to our theoretical knowledge on the mechanics of sedimentation. The use of the "distribution function" seems to be the correct attack to account for the variation in settling velocities of particles in an unsettled suspension. It is going to be very difficult to express mathematically the effects of coagulation. Unfortunately, the sanitary engineer seldom has to deal with granular sediment but sediment that coagulates and changes its hydraulic properties during settling.

The author's suggested method for determining the turbulent velocity is applicable only to granular material. We have all observed a nice floc in a basin and on attempting to get a sample in a beaker we find it very fragile. The fragile floc has value in sedimentation, but it is difficult to measure.

Professor Slade concludes "that the design of settling tanks may be placed on as precise a foundation as that, for instance, on which rests the design of trusses." I am afraid that it will be possible only with granular materials, but not with materials which tend to coagulate. Nevertheless I feel Professor Slade has added much to our fundamental knowledge of sedimentation, and that much of practical value can be developed from the start that he has given us.

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(1) MORRILL, ARTHUR Sedimentation Basin Research and Design. J. A. W. W. A. **24**: 1442 (1932).
(2) HAZEN, ALLEN On Sedimentation. Trans. A. S. C. E. **53**: 43 (1904).
(3) SLADE, J. J., JR. Sedimentation in Quiescent and Turbulent Basins. Proc. A. S. C. E. **61**: 1435 (1935).
(4) PEARL, JAMES W. Lateral Sedimentation. Private pub., 1932.

*Discussion by* GORDON M. FAIR:* Professor Slade has attacked the difficult problem of sedimentation in a new and ingenious manner.

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Engineers will regret that he has not found time or opportunity to apply his analytical approach to experimental investigations. While Professor Slade proceeds from a different point of view than that taken by Allen Hazen in his classical paper "On Sedimentation" the ultimate conclusions reached are strongly convergent. Professor Slade's approach to the problem awaits empirical determination of the turbulent velocities of settling tanks. Hazen has given an answer in terms of serially subdivided flow.

The design of coagulation basins for water purification plants involves factors other than those entering into the problem of sedimentation. Coagulation basins are first of all reaction chambers in which a proper floc must be formed with as small an amount of coagulating chemical as is economically consistent with basin size. Coagulation basins must hold the water that is undergoing treatment until the necessary reactions are completed. These reactions or interactions are promoted by gentle stirring and the factors that induce coagulation are generally contrary to those that satisfy sedimentation. Only after coagulation is completed is there a need for sedimentation. If we are to continue the practice of asking the entrance portion of a coagulation tank to serve as a mixing chamber and the effluent portion to act as a settling chamber, we must at least recognize the different functions of these two portions of coagulation basins and plan our designs accordingly. Ordinarily the settling properties of well-coagulated water are such that removal of floc can be accomplished in a short period of subsidence.

The question that the writer should like to raise in connection with this discussion of sedimentation, therefore, is whether the time has not come to recognize more clearly the different functions that coagulation basins serve and perhaps to allocate these functions separately to two units:—a reaction tank or true coagulation basin and a sedimentation tank.

*Closing Discussion.* By J. J. Slade, Jr.: The writer agrees with Professors Carpenter and Fair that the problem of sedimentation is complex, but he feels that little is gained by reiterating this fact. He seeks rather to analyze the problem into mutually independent elements each of which possesses its own type of simplicity. In the present paper the writer has attempted to formulate the problem of sedimentation as such, which is only one aspect of the general problem presented by sedimentation basins. Up to a point this formulation is independent of the rest of the problem, but the problem as a whole depends on a clear understanding of it.

The writer disagrees with Professor Carpenter that this formulation is valid only when the suspended material is granular. In a statistical investigation, which this problem with its numberless variables clearly requires, the measurable quantities appear only as distribution functions, arithmetical means, standard deviations, etc., and, so long as they can be measured, their physical nature is irrelevant. The only condition necessary for the validity of the analysis is that the solid material, whatever its nature, be stable during the

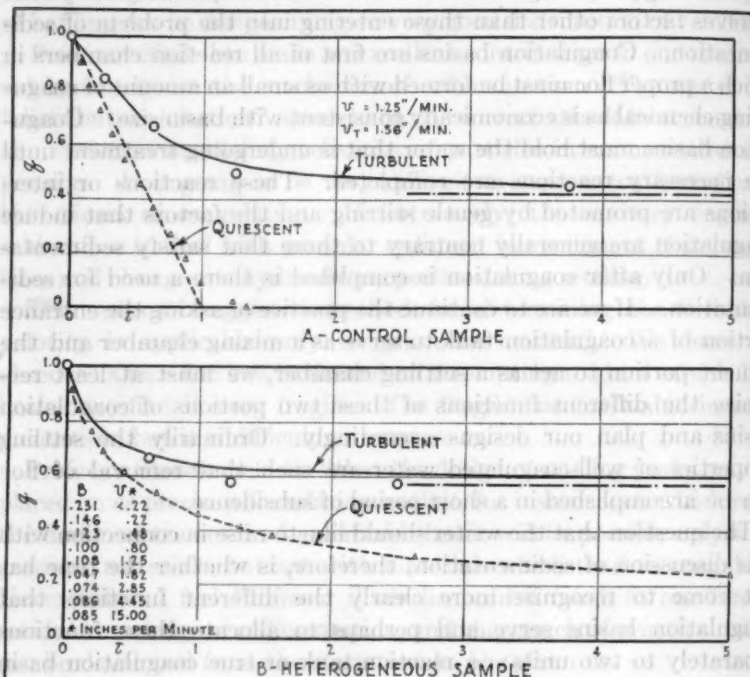


FIGURE 8

process of sedimentation. Even when the floc changes its hydraulic character with time, the general analysis is not invalidated, but in this case the tables cannot be applied without modification.

The writer's work suggests two series of studies. In the first place flocculation should be studied with regard to hydraulic value distribution. It is not sufficient to know that under certain physicochemical conditions a good floc is precipitated at the end of fifteen minutes. It is necessary to give this "goodness" a measure; that is,

it is necessary to determine the hydraulic value distribution and its stability under various degrees of turbulence. In the second place basins of all shapes and sizes should be studied with regard to the turbulent energy they impart to the fluid at various rates of discharge and under various modes of operation. It is only when these two series of studies have been carried out to a great degree of completeness that they can be combined to show quantitatively what happens in an actual basin and the allocation of functions made that is suggested by Professor Fair.

The general validity of the writer's theory of sedimentation may be easily verified experimentally without elaborate equipment. In a series of preliminary experiments the writer used powdered quartz. By fractional decantation he obtained a sample with a fairly uniform hydraulic value of 1.25″/min. Since he used no peptizing agent, there was a certain amount of coagulation, as is shown by the triangular points in figure 8 (A). The dotted line in this figure represents the settling curve for 1.25″/min. which he adopted as the average for this sample in still water. He placed some of this control sample in a beaker of water and by pouring from one beaker to another he obtained a uniform mixture. He allowed this mixture to settle while stirring the water with a pencil with a figure-8 motion at a speed he was able to maintain uniformly for an indefinite period of time. Periods of retention of 1, 2.5, 5, and 15 minutes were used. At the end of these periods the liquid was poured off and the amount remaining in suspension measured by means of a hydrometer. The average results of three runs for each period of retention are given by the circles in figure 8 (A).

The first problem was to determine the turbulent velocity imparted to the water by the stirring. Since we have the amount of material remaining in suspension (45 percent) at the end of 15 min., $v =$ 1.25″/min., and $h = 5''$ (the height of the beaker), we readily find from the tables that the turbulent velocity of the fluid is 1.56″/min. (approximately, because no interpolation was used). The solid line of figure 8 (A) is the curve obtained from the tables for the above values of $v$ and $v_T$. The agreement with the observed results is seen to be very good.

The writer next took an unprepared sample of the quartz and determined its settling curve in still water to be the dotted line of figure 8(B). From this, by the method outlined on the Proceedings paper, he determined the distribution of hydraulic values, which are listed

in the same figure. As with the control sample he observed the heterogeneous sample settle out under the same conditions of turbulence ($v_T = 1.56$) with the result given by the circles in the figure. The theoretical curve, obtained from formula (9), is given by the solid line. The agreement is seen to be fair; if certain corrections suggested by the general theory are made (such as taking into account the vertical distribution, since some liquid was necessarily left in the beaker), the agreement is greatly improved, but the crudity of the experiments does not warrant any arithmetical refinements. The writer expects later to report the results of a series of more systematic and better controlled experiments he has under way.

# ABSTRACTS OF WATER WORKS LITERATURE

**Key: JOURNAL of the American Water Works Association, 29: 10 (1937). The figure 29 refers to the volume, 10 to the page of the JOURNAL and (1937) to the year of issue. If volumes of a publication are not paged consecutively but by issues, the figures 29: 1; 10 (1937) indicate—volume 29, number 1, page 10 and 1937 as the year of issue. Initials *W. P. R.* signify that abstract is reproduced, by permission, from *Water Pollution Reports (British)*.**

## CONCRETE

**Resistance of Concrete to Chemical Attack.** J. M. ANTILL. The Commonwealth Eng., 24: 395, July 1, (1937). Limitations of concrete sometimes overlooked. First consideration must be of strength but consideration must also be given to degree of permanence. Concrete exposed to ground water, natural waters, domestic sewage, trade wastes, numerous oils, fats, acids, etc. The chemical attack of concrete occurs through a reaction between the constituents of the set cement and the particular destructive agent concerned; generally speaking the aggregate itself remains unchanged. Water tightness and density are therefore of importance. Cements considered are Portland, aluminous, supersulphate, blast-furnace and permetallurgical. Ave. constituents given in table, all give different final chemical end products to which the inherent resistance to chemical attack is intimately related, particularly to the quantity of free hydrated lime liberated. In Portland cement this free lime remaining after hydration is no less than 10% by wt. of cement, blast furnace cement may contain a little, permetallurgical usually none at all. Aluminous cement produces hydrated compounds of alumina and lime together with free hydrated alumina which is inert to most of the destructive agents. Through similar reactions protection is secured in permetallurgical and blast-furnace cements. Various factors control vigor and extent of attack on concrete by sulfate waters. In general these salts react with the hydrated calcium aluminates, forming calcium sulphoaluminates which occupy a greater volume than the original materials and therefore cause expansion with subsequent disruption. The uncombined calcium hydroxide also reacts with the sulfate salts other than calcium, to form calcium sulfate which in crystallizing in the concrete causes expansion and disintegration. Concrete so attacked is finally reduced in a majority of cases to a soft mush, but when the aggressive water contains a predominating amount of sulfate salts, the disintegrated material is usually harder and of a granular appearance. Can usually be recognized in earlier stages by general softness and presence of white deposits in the concrete. Presence of water essential to sulfate attack. Most dangerous condition arises when one surface of the concrete is exposed to the atmosphere and one surface is exposed either to sulfate-bearing water or soil containing sulfates. Aluminous cements through long experience have been proven to be

immune to attack by sulfate waters, but require expert handling for placing and are expensive. Supersulphate cements without need of extra precaution of placing, are less expensive and are satisfactory and have all the good qualities of aluminous cements. Puzzolanic (blast-furnace, permetallurgical and similar) cements, while not as immune as 2 above are much better than Portland cement. Free sulfuric acid even in very small quantities is extremely deleterious to concrete. Deterioration, in some cases within 2 yrs., of concrete pipe has been reported due to $CO_2$ in solution. Pure water has a solvent action only and does not endanger structure because action is on the surface in which free lime is removed. In Norway and Sweden however, several large dams have given trouble, the leaching away of lime having become so pronounced as to cause considerable leakage through the dam. Caution should be observed with waters contg. as low as 100 p.p.m. of total solids and pH below 7. Discussion is also given of the affect in sewers and with trade wastes. —*Martin E. Flentje.*

**Construction of the New Britain Water Tower.** J. W. Holden. Conn. Soc. Civ. Engrs., p. 112 (1937). Detailed description of method of constructing a 1,500,000-gal. standpipe, being built at New Britain, Conn. This standpipe is of particular interest because it involves use of new methods of concrete reinforcing and also use of gunite as a structural medium. Design of New Britain tank is not based on principle that concrete and steel must be bonded together in order to get proper results. Special asphaltic coating not only prevents bonding of concrete to steel but also serves as lubricant which frees the rod to act independently of the concrete.—*G. C. Houser.*

**Workability of Concretes and Mortars.** Edw. W. Scripture. Eng. News-Rec. 119: 17 (1937). A discussion of the desirable qualities of concretes and mortars for various purposes, the 2 factors which determine workability, i.e., mobility and cohesiveness, and the effects of the mix components and admixtures on these properties. In mixes lacking in fines, addition of either inert or puzzolanic fines is advantageous; in other mixes such additions are detrimental. A number of materials of a colloidal nature are available which greatly increase workability without interfering with design of mix and improve the properties of the finished concrete. Eight references.—*R. E. Thompson.*

## CORROSION CONTROL

**Initial Corrosion Rate of Mild Steel—Influence of the Cation.** C. W. Borgmann. Ind. Eng. Chem., 29: 814 (1937). Tests on cold rolled strip steel in aqueous solutions at 35°C. showed that in neutral solutions in which anion is non-oxidizing and non-reducing and forms soluble corrosion products, corrosion rate depends on nature of cation. In salts of alkali metals and of alkaline earths, tendency to corrosion decreased with decreasing atomic weight under conditions of test. In solutions of salts acid by hydrolysis (ammonium, ferric, aluminum and chromic chlorides) cation exerted considerable effect on corrosion rate, and attack proceeded with evolution of hydrogen or reduction of cation of salt. In mixed chloride solutions, addition of small amounts of

one salt to another tends to lower corrosion rate, and frequently affects nature and distribution of attack. A few tests on zinc in chloride solutions showed that here too cation has important part in determining corrosion rate.—*Selma Gottlieb.*

**Corrosion Control by Deaeration.** S. T. POWELL AND H. E. BACON. W. Wks. and Sew. 84: 109 (1937). Despite various orthodox chemical treatments, corrosion of 9-mile pipe line caused 40% loss of carrying capacity. Deaeration was tried experimentally, and on basis of experiments, was put into service. Results have been entirely satisfactory. Deaerator is described.—*H. E. Hudson, Jr.*

**Some Comments on Corrosion in Water Works Practice.** CARL N. ALEXANDER. W. Wks. and Sew. 84: 99 (1937). A general discussion of corrosion factors including acidity, dissolved oxygen, carbon dioxide, pH, protective films, galvanic action, salt concentrations, and micro-organisms. External corrosion and applied coatings are touched upon.—*H. E. Hudson, Jr.*

**Corrosion of Water Mains.** ANON. Surveyor. 92: 106D (1937). The atmosphere, the soil, and the accidental infiltration of corrosive liquid or marsh water may affect the external surface of a pipe while the causes of internal corrosion may be mineral acidity, dissolved oxygen, carbon dioxide, soluble salts, galvanic effects, or certain microorganisms. An impermeable insulating coating is desirable because it not only retards corrosion, but also prevents pitting. Soluble salts, such as common salt or sodium sulphate or magnesium chloride are excellent electrolytes and increase the corrosion of iron when found in very dilute form. Certain microorganisms such as crenothrix are responsible for corrosion troubles. With regard to soil corrosion underground pipes are generally made of cast iron which is more resistant to corrosion than steel or iron. Refined tar pitch is mentioned as containing very powerful corrosion inhibitors. Portland cement coatings are used where conditions are very severe such as in marshy places and sometimes bituminous coatings are followed by cement coatings.—*H. E. Babbitt.*

**Rustproofing Fort Peck Penstock.** ANON. Eng. News-Rec. 119: 115 (1937). One of 4 outlet tunnels of Fort Peck Dam was lined with steel pipe for distance of 3,118′ downstream from control gate shaft to provide 24′-8″ penstock for possible future power development. Steel lining was cleaned by steel grit blasting, sprayed with cold priming coat of coal-tar paint, coated with hot coal-tar enamel with dauber brushes and finally tested for flaws in coating with electric broom. Painting operations were serviced by electrically-heated melting kettles of special design that supplied enamel at temperature of 470°F. Electrically-heated paint pots were also used. Total area covered was 242,000 sq. ft.—*R. E. Thompson.*

**Tuberculation in Reverse.** I. M. GLACE. W. Wks. and Sew. 84: 210, (1937). A brief resumé of knowledge of corrosion is given. At Royersford Pa., after correcting aggressiveness of previously active water, it was observed

that tuberculation stopped, and that apparently the iron rust sloughed off the pipe interior, causing many consumer complaints. Difficulty was experienced in maintaining pH in distribution system.—*H. E. Hudson, Jr.*

## DAMS

**Flood Control for London Reservoir.** ANON. Civ. Eng. (Br.) 32: 298 (1937). A 5-siphon spillway has been provided for the Brent reservoir situated in the urban districts of Hendon, Willensden and Kingsbury. Water enters each siphon through an opening of approximately 82 sq. ft. at a depth between 7′ 6″ and 8′ 0″ below the reservoir level. The vertical height from crest to pool level is approx. 30′. The air inlet pipes which are connected to the crown of each siphon are 15″ in diameter. The siphons are designed to discharge 600 c.f.s. Tests were made with a model before construction of the structure.—*H. E. Babbitt.*

**Discharge of Needle Valves and Sluiceways at Madden Dam.** P. S. O'SHAUGHNESSY. Civ. Eng. 7: 576 (1937). The regulatory works at Madden Dam include two outlet pipes equipped with needle valves, and six rectangular sluiceways controlled by gates. Tests have been made to determine the actual head-discharge relationship for both of these appurtenances. By using "group" pitometer readings in preference to individual readings the field work could be done in much less time, and the office work would be reduced considerably because the graphical determination of the discharge would be unnecessary. Mechanical tests were made to determine the amount of torque required to close the butterfly valve. At the highest head tested (114.2′) it was found that the motor torque remained practically constant at 20 ft. lbs. from the start of the operation to 60% closure. Seating of the valve increased the motor torque to 48 ft. lbs. equivalent to a torque of 1,344,000 ft. lbs. at the valve stem. Sluiceway tests covered the range of heads between 71.5′ and 138.5′. At the highest head the mean velocity through the sluiceway was 72.4 ft. per sec. Some data have been obtained on sluiceways operating at partial gate openings, but they are not complete enough to develop satisfactory conclusions. Head-discharge curves for both the valves and sluiceways are given.—*H. E. Babbitt.*

**The Gebel Aulia Dam.** ANON. Civ. Eng. (Br.) 32: 280 (1937). The dam is located on the White Nile about 25 miles south of Khartoum. The structure is 5,000 meters long, including a masonry section of 1,693 meters and an earth embankment of 3,305 meters. Sixty sluices and a lock are included in the masonry section. The masonry section is of granite throughout while the earth section is compacted fill faced with mosaic sandstone pitching. The dam is surmounted by a roadway open to traffic. The underlying formation on the site consists of a somewhat friable sandstone into which a continuous cut-off wall was tied. Tests on models preceded the final design of the structure. Each sluice gate consists of a built-up steel structure formed of rolled steel sections with skin-plating on the upstream side, all riveted together. Operation of the gates is by means of independent spur-geared headstocks. The lock section is approx. 60 meters long by 18 meters wide. A fish ladder

has been provided. The length of the reservoir which will be formed is estimated to be over 300 km. with a max. width of over 7 km., giving a storage capacity of 3,500,000,000 cu. meters covering an area of 1,752 sq. km. The reservoir was closed for filling in July, 1937.—*H. E. Babbitt.*

**The Chambon Reservoir, Isere, France.** THEODORE RICH. Eng. **144:** 167 and 219 (1937). The reservoir serves to regulate the flow of the Romanche River for the improvement of the operation of a 140,000 kw. power plant. The river is glacier fed, there are heavy floods in the spring, and violent floods of short duration in Aug. and Sept. following thunderstorms; in the winter the flow may fall to a very low figure. The variations of flow were between 1.5 and 80 cu. m. per sec. The result of the construction of the dam has been to increase the min. flow to 5.7 cu. m. per sec. The dam is of the gravity type and has a length of 293.6 m., a height above the foundation of 136.7 m.; the greatest height above the stream bed being 91.2 m. The thickness at the top is 5 m. and at the foundations is 69.7 m. The foundation was placed by the use of cofferdams which were used for depths of 42 m. below the river level. Topographic indications were that the gorge selected provided an ideal dam site but the foundation work revealed the existence of two glacial mills with pot holes not shown by the original borings. This difficulty led to much expense in the excavation for the foundations and a revision of the design of the dam itself. Although the rock was in general excellent as to impermeability a complete cemented screen was formed across the whole site. 412,000 cu. yd. of concrete were used in the construction of the dam, which includes large plums of stone which occupy 8% of the mass. The dam is constructed in 20 sections varying in width from 11 to 22 m., and it is provided with a complete system of drainage conduits. The useful capacity of the reservoir is 54,000,000 cu. m.—*H. E. Babbitt.*

**Gene Wash and Copper Basin Dams.** ANON. Eng. News-Rec., **118:** 763 (1937). Unit prices of 3 lower bidders are given. Both dams, located at east end of Colorado River aqueduct near Parker Dam, will be thin concrete arches. Gene Wash arch section will be 156 feet high with crest 315 feet long, joining gravity section 115 feet long. Earth dikes, of 25-foot maximum height and 1300 feet long, will be required. Copper Basin Dam will be 210 feet high and 220 feet long. Cooling coils of 1-inch tubing will be embedded in concrete of both dams.—*R. E. Thompson.*

**Marshall Ford Dam Designed for Raising in Future.** Eng. News-Rec., **118:** 712 (1937). Marshall Ford Dam, being built by Bureau of Reclamation on Colorado River, 22 miles upstream from Austin, Texas, is designed for 2-stage construction. Present plans call for straight gravity structure 190 feet high and 2325 feet long, with 730-foot overflow spillway at present river channel. Left abutment will be flanked by rolled-earth embankment 1100 feet long, 30 feet in maximum height and faced on both sides with rock. Ultimately, spillway section will be raised 74 feet and non-overflow section 75 feet, giving dam maximum height of 265 feet. Earth embankment will be raised accordingly. Diversion will be effected through two 26-foot conduits built into left end of spillway section.—*R. E. Thompson.*

**Boulder Dam Earthquakes Continue.** Anon. Eng. News-Rec. **119:** 178 (1937). Earthquakes at Boulder Dam continued through first half of 1937 and, although no serious damage resulted, frequency of shocks now occurring where none were reported before dam was built has given rise to much speculation as to whether cause lies in new load placed on earth's crust by Lake Mead. Two residents who have lived in vicinity for 15 and 17 years report never having experienced earthquake prior to completion of dam. United States Coast and Geodetic Survey now has 3 accelerographs in or near dam and 2 more installations, one on either side of Lake Mead, are proposed. Between September 7, 1936, and July 1, 1937, 49 earthquakes were reported or recorded. —*R. E. Thompson.*

**Contractors Win River Battle.** Anon. Eng. News-Rec. **119:** 13 (1937). Report on repair operations on Grand Coulee Dam cofferdam appeared in April 22 issue of Eng. News-Rec., p. 595. In this article, additional data are given regarding conditions encountered and remedial measures adopted. For a time, inflow was 35,000 g.p.m. Complete stoppage was effected after strenuous struggle carried on in face of serious risk of flooding east half of foundation where concrete pouring is now under way. Cells in both up- and downstream cofferdam arms were reconstructed and it is now believed that no flood likely to occur this summer will menace unwatered area.—*R. E. Thompson.*

**New Record in Pouring Concrete.** Anon. Eng. News-Rec. **119:** 101 (1937). About half the concrete in present or "foundation" contract at Grand Coulee Dam had been placed by May 15. Experiences in preparing east half of river bottom for concrete and in tuning up equipment for record-breaking concrete placement program that is to complete initial stage are outlined. Horizontal grids of cooling pipes, through which river water at temperature of 50°F. is circulated, are laid on top of each 5-foot lift of concrete. Temperature of water rises 16° in 10-15-minute period of circulation.—*R. E. Thompson.*

**Missouri River Blocked at Fort Peck.** C. H. Chorpening. Eng. News-Rec. **119:** 153 (1937). Missouri River at Fort Peck Dam was diverted on June 24 from old channel into 4 concrete-lined tunnels through shale bluffs at east side of valley by closure of center opening of dam. Plans for closure at upstream toe on June 26 were suddenly changed by unexpected slide of river bank at downstream toe which threatened loss of railroad bridge at that point. Closure was successfully made under emergency conditions. Details given.—*R. E. Thompson.*

**Thin-Core High Earth Dam.** Orman M. Strange. Eng. News-Rec. **119:** 195 (1937). Dam under construction on Ralston Creek will be one of highest earth dams in world, made up of 2,400,000 cu. yds. of rolled fill protected by riprap face coverings and rockfill toes. Exceptional care in construction is required because of relatively high clay content of material used in core, which is of unusually thin cross section. A soils laboratory has been established on the site. Reservoir, 20 miles northwest of Denver, will serve as storage

and distribution unit for part of water Denver is diverting from West Slope of Rocky Mountains through pioneer bore of Moffat tunnel into South Boulder Creek. Conduit system of canals and tunnels carries water from South Boulder diversion dam along foothills to new reservoir. Diversion is possible only in summer months, which necessitates storage to carry through winter. Reservoir of at least 10,000-acre-foot capacity was desired. Although it lacked higher capacity available elsewhere, Upper Ralston Creek site was selected because elevation was such as to allow gravity flow through West Side filters and capacity was sufficient to provide water for city's high areas during winter, eliminating use of Ashland Ave. pumping station and thereby effecting substantial operating saving. Crest of dam is 25′ wide and 1150′ long. Tunnel, 950′ long, takes care of creek diversion during construction and will form controlled outlet works for reservoir. For 260′ from inlet portal, tunnel is of circular concrete-lined pressure section, 10′ in dia.: remainder is non-pressure, 10′ horseshoe section, heavily reinforced to resist possible ground pressure and carrying 60″ steel pipe discharge line. Combined capacity of 2 spillways and outlet works, which are described, is 32,000 sec. ft., double previous maximum flow of creek. Completion is expected this fall.—*R. E. Thompson.*

## FLOODS

**Gaging the Rivers of Connecticut.** B. L. Bigwood. Conn. Soc. Civ. Engrs., p. 14 (1937). First part of paper relates to methods employed in river measurements and contains general description of equipment used and methods followed in field and office in collection, compilation and publication of stream-flow records. Map showing location of gaging stations in Connecticut is included. Second part of paper describes the great floods of March, 1936, and discusses climatologic and meteorologic phenomena prior to and during the floods, followed by review of gaging station operations under the unusual conditions that prevailed at those times. Tabulation of maximum flood flows is included.—*G. C. Houser.*

**Flood Sanitation Problems.** H. K. Gidley. Municipal Sanitation, 8: 273 (1937). West Virginia encountered two record breaking floods during a ten month period. High water occurred almost overnight and created numerous emergency public health problems. Such emergencies may find health organizations unprepared, unless definite information on such sanitation problems is available. Author discusses problems that may be of interest to others likely to be faced with similar situations. Emergency Excreta Disposal: Failure of the central water supply will cripple the sewerage system and may necessitate the use of primitive means of body waste disposal. At the crowded refugee centers sanitary earth pit privies were erected. For the large number of persons gathered in the high portion of the business district it was necessary to erect toilet facilities over man holes along the streets. Refugee Stations: Control discussed. Author also discusses: disposal of garbage, control of flood damaged foodstuffs, sanitary inspection of flooded food-handling establishments, milk sanitation, and sanitation in rural areas, including treatment of private wells contaminated by flood waters.—*R. E. Noble.*

**Cloudburst Flood Formulas.** C. C. Inglis. Eng. News-Rec. 119: 48 (1937). In 1930, writer, in collaboration with A. J. DeSouza, wrote Technical Paper No. 30 of Government of Bombay Public Works Dept. entitled, "A Critical Study of Runoff and Floods of Catchments of the Bombay Presidency." Gaging was carried out on rivers with catchments varying from 50 to 24,000 sq. mi. Discharges frequently had to be calculated by measuring slope from flood marks, assuming a rugosity value, ignoring effect of supercharge of bed silt, and assuming same section as before or after flood had passed. It was found that: (1) Observers had strong tendency to over-estimate phenomenal floods. (2) Slope measurements were liable to considerable error. (3) Surface slope often exceeded bed slope and so was not true measure of energy gradient, on which discharge depends. (4) Changes in river section occurred during flood. (5) Loss of energy resulted from standing waves, especially in hill torrents. From the Indian data, writer worked out formula: $Q = 7{,}000 \sqrt{A}$, where Q = discharge in second-feet and A = area of catchment in square miles, which, when written as run-off per sq. mi. of catchment, becomes: $q = 7{,}000 \sqrt{A}$. Although suitable down to catchments of about 10 sq. mi., this formula is unsuitable for smaller catchments; by modifying it to $q = 7{,}000 \sqrt{A} + 4$, it is applicable to all sizes of catchments.—*R. E. Thompson.*

**New England Flood Plans Analyzed.** Anon. Eng. News-Rec. 119: 144 (1937). Interstate compacts for flood prevention in Connecticut River Valley have been approved by legislatures of Connecticut, Vermont, Massachusetts and New Hampshire and in Merrimack River Valley by latter 2 states. Only formal approval of Congress is required before agreements become effective and construction can be started. Data are given from recent paper by H. K. Barrows in which it was pointed out that agreements follow recommendations of Water Resources Committee of National Resources Committee and comparison was made between latter recommendations and those of U. S. Engineers for the 2 basins. Problem was taken up in broad way following flood disaster of March, 1936. Flood in Connecticut Valley at that time resulted in damage of about $35,000,000 and was of order of 300-500-year frequency. Committee held that solution of problem lies in use of reservoirs primarily for power and incidentally for flood prevention, in harmony with recreation and sanitation.—*R. E. Thompson.*

**Checking Torrential Floods.** Anon. Eng. News-Rec. 119: 22 (1937). Flood control problem in Los Angeles County is one of most difficult in country. Since 1914, year of disastrous floods, over $64,500,000 has been expended under direction of Los Angeles County Flood Control District. On application of District, federal grant was made in 1935 for 10 specific items of control works on condition that work be executed by War Department under direction of Chief of Engineers. Work previously planned and carried out by local financing and administration is reviewed. In general, program contemplates taking peaks off floods by storage reservoirs at foot of mountain slopes which constitute northern portion of district's area and, wherever possible, releasing flood waters in such a way as to replenish underground supply. Excellent results have attended replenishing operations, total of 650 acres of spreading

grounds being operated by District. During 1937 spring runoff, 100,000 acre-feet were released from reservoirs and spread at rates that would permit percolation into underground basins. Fourteen storage dams have been completed and are in operation and another, San Gabriel No. 1, is approaching completion. Channel improvements have included straightening and lining (where necessary) of natural stream beds. More than 70 miles of permanent channel improvement has been constructed and in addition about 140 miles of channel has been protected by temporary improvements. (See following abstract.)—*R. E. Thompson.*

**Structures to Control Torrents.** R. E. CRUSE. Eng. News-Rec. 119: 67 (1937). When army engineers took charge of Los Angeles County flood control work (cf. previous abstract), District had assembled all known records of rainfall, runoff and related matters, set up gaging stations and developed comprehensive plan for control of floods, part of which had been completed. WPA funds were restricted to 10 specific items selected by District as most important and urgent items in comprehensive plan. These items are described. The structures divide into 2 main classes: (1) Débris basins and dams at mouths of mountain canyons, with outlet channels leading to main river drainage system, and (2) channels for enlargement and improvement of certain portions of Los Angeles River and its principal flood-producing tributaries, and Ballona Creek. Excavation will total 14,105,000 cu. yds. and concrete 502,600. Details given.—*R. E. Thompson.*

**Unusual Flood in Ohio Town.** ANON. Eng. News-Rec. 119: 87, 116 (1937). Heavy rainfall occurred to south of Bellevue on June 26, reaching cloudburst proportions in neighborhood of city. It is estimated that 7″–8″ of rain fell between 3 a.m. and noon. Flood waters backed up behind railroad embankments south of city to depth over farmland of 20′–30′ in places. Underpass through embankment was of insufficient size to permit release of waters, necessitating breaking open embankment. Embankment usually protects city from floods. Streets in low areas became rivers flowing to depth of about 4′, great number of houses were flooded at first floor level and 90% of basements were filled with water. There are no surface streams in vicinity of city and normally drainage disappears into subterranean caverns, which are also used for sewage disposal. In some instances, flood resulted in upward flow from these caverns and in several places geysers 15′–20′ high broke through streets. Water supply is obtained from 3 upland reservoirs filled from small surface ditches during flood flows. Modern purification and softening plant was placed in service few months ago. Plant was not flooded but water backed up around buildings to within 3″ of elevation of filter room floor. Wash water normally flows into artificial sink-hole discharging into subterranean caverns and as latter were flooded to within 1′ of ground surface and there was no other way of disposing of wash water it was necessary to bypass filters during period of flood. Residual chlorine content of 0.3 p.p.m. was maintained in distribution system. Very serious sanitation problem resulted from backing up of sewage.—*R. E. Thompson.*

**Flood Control by Reservoirs.** ARTHUR E. MORGAN. Eng. News-Rec. 119: 263 (1937). The Jadwin plan of 1928, which was adopted following disastrous flood in 1927 on lower Mississippi River, was based on enlarged levee system with relief floodways at critical locations designed to come into operation during maximum floods. Actual flood heights below Cairo during 1937 flood show that system will not safely carry design quantity of water. Even in reach from Cairo to below Helena, where record heights were experienced, 1937 flood cannot be considered as the greatest possible, as discharge was largely from Ohio River. This means that more protection must be obtained. Most promising aid available is reservoir control. Combination of reservoirs and levees, with help of spillways and revetments and occasionally other methods, appears to offer best solution of problem. Tennessee Valley Authority has made preliminary study of possible flood control on Ohio River near Paducah. One project consists of dam on Tennessee River at Gilbertsville and dam across Ohio and Cumberland Rivers at Dog Island, with short interconnecting navigation and equalizing canal between Tennessee and Cumberland joining the 3 rivers into common reservoir pool. Second project consists of single large dam across both Tennessee and Ohio Rivers at mouth of former just above Paducah. Investigations thus far completed indicate former scheme to be preferable. Advantages which would accrue are dealt with in some detail.—*R. E. Thompson.*

## HEALTH AND HYGIENE

**The Fluoride Content of North Dakota Ground Waters as Related to the Occurrence and Distribution of Mottled Enamel.** G. A. ABBOTT. N. Dak. Geological Bull., No. 9, (1937). Bulletin describes studies started in 1929 of waters in N. Dak. which appeared to have bearing on mottled enamel in teeth of inhabitants of certain communities. Lesions of mottled enamel show progressive stages of severity, from effects so inconspicuous as to escape recognition even by many dentists, through increasing degrees of severity from appearances of occasional paper-white spots, mild pitting and corrosion; to conditions involving the whole tooth structure accompanied by disfiguring brown and black stains and severe pitting and corrosion. In severe cases hideous malformations of the teeth are often observed, and the whole tooth structure is hopelessly destroyed. A nation-wide survey by U. S. Public Health Service demonstrated an area where mottled enamel is endemic and severe in northeastern S. Dak., extending into N. Dak. in Richland, Sargent, Dickey, and some other counties. Malady reported to be especially common and severe near Lidgerwood and Forman, also portions of Ransom, LaMaure, Cass and Traill counties. Since 1934 over 800 mineral analyses of water supplies made. These results when compared with information of dental surveys showed that correlation was complete and convincing; in every instance where mottled enamel was reported as general and severe the water showed high fluoride values. Throughout whole region under study the severity of the malady was found to follow contours of fluoride map. Such a contour map is included showing fluoride content of water from the Dakota Sandstone and Red River Valley Artesian basins. In general, it was found most deeper ground waters carried sufficient fluoride to cause mottled enamel. Fluorides

in water may also have other pathological effects such as alteration of bone structure, and causing enlargement of thyroid gland, and may result in serious disturbance of body metabolism. Fluoride removal is not now practical and several commercial units designed to do this when tested did not function very successfully. Typical analyses of some fluoride bearing waters are given.—*Martin E. Flentje.*

**Hygiene of Drinking Water.** O. SPITTA. Gas-u. Wasserfach. **80:** 298 (1937). The author shows that only few diseases are transmitted by water and that at present these diseases are much rarer than other common infectious diseases. Yet a reduction of the sanitary care given public water supplies would be dangerous. It is true that the connection of recent typhoid epidemics with the local water supply as to their cause can only be proven indirectly, owing to the long period of incubation and to the difficulties of detecting typhoid organisms in the water. The reasons for such an explanation are given and show that it is much clearer than an explanation according to Pettenkofer's soil theory. This latter claimed that the epidemics were due to "sick soil" which under the influence of combination of outside factors has gassy emanations that cause the sickness. Spitta takes a strong stand against this old theory, which has lately been defended by Wolter. Yet he admits that there are still many points in typhoid epidemics unexplained, like their yearly and local distribution, their connection with the water sickness which precedes many of them, the number and type of organisms necessary to cause sickness. But based on his interpretation of the facts observed at such epidemics he believes that more careful filtration with less dependence on chlorination could give further improvement.—*Max Suter.*

**Certain Public Health Bills Passed by the 1937 Connecticut Legislature.** ANON. Conn. Health Bulletin, **51:** 165 (1937). Among bills passed by Connecticut legislature in 1937 is an act which provides that any one operating a laboratory, in which water or sewage analysis is made a basis for advice as to sanitary quality of such water or sewage, shall be subject to fine unless the laboratory has been approved by state department of health. Another act provides that the department shall have jurisdiction over all matters concerning purity of any source of water or ice supply used by any municipality, public institution or water or ice company. An act was passed authorizing the department of health to make rules and regulations for protection of the purity of interstate waters used as sources of drinking water supply. The legislature also made provision for appointment of a commission to act jointly with similar commissions of New York and New Jersey in formulating a treaty to prevent and eradicate pollution of waterways common to 2 or more of the 3 states.—*G. C. Houser.*

**Diarrhea Epidemic in Dallas.** ANON. Eng. News-Rec. **118:** 648 (1937). Between April 8th and 14th, about 3,000 residents of Oak Cliff section of Dallas, Texas, were stricken with diarrhea. All cases occurred among population of about 10,000 served by one water main. Laboratory tests indicate that epidemic was due to a dysentery organism which was recovered from supply

main and from stools of patients. Water samples invariably showed much gas and low bacterial count: presumptive positive fermentation tubes occurred within 8-12 hours. Source of contamination has not been located. No typhoid cases have been reported to date. Supply of Oak Cliff section, which has never been chlorinated, is obtained from artesian wells, temperature of water being 108°F. Since discovery of contamination, water has been treated with heavy dosage of chlorine. Water supply of Dallas proper is derived from surface sources and is chlorinated.—*R. E. Thompson.*

## HYDRAULICS—HYDRAULIC ENGINEERING

**Charts and Slide Rule for Pipeline Calculations.** ANON. Eng. 144: 135 (1937). In pipeline flow problems when dealing with full-bore flow there are four factors to be considered: the properties of the fluid, the pipe diameter, the rate of flow, and the loss of head. A formula which will care for these factors is presented and its application to charts and slide rule design is explained.—*H. E. Babbitt.*

**Flow of Fluids Through Openings in Series.** F. DOLLIN AND W. S. BROWN. The Engr., 164: 223 (1937). The problem of calculating pressure-quantity relationships for the flow of a fluid through a number of openings arranged in series is one that is fundamental in turbine design. The problem is analyzed mathematically and a formula is presented by which the quantity of discharge can be computed in terms of measurable conditions.—*H. E. Babbitt.*

**Flow on Steep Slopes.** JOHN HEDBERG. Civ. Eng. 7: 633 (1937). Practically all our hydraulic theory on flow in open channels carries the proviso that flat slopes are assumed. Yet some of our most troublesome problems are concerned with chutes on grades of 10 to 30%. Flow on steep slopes is distinctly a different problem from that on flat grades. Either it should be treated as such, with the necessary refinements in theory, or the errors of the simpler theory should be recognized by increased factory of safety.—*H. E. Babbitt.*

**Model Study of Spillway Characteristics.** J. J. DOLAND, T. E. LARSON, AND C. O. REINHARDT. Illinois State Water Survey. Circular No. 20 (1937). By using the head-discharge curves obtained from the model studies it was expected to make use of the hydrologic data that is being collected on the West Frankfort Reservoir. The danger of extrapolating quantitative results of model studies to the performance of the prototype is recognized. However, at the present time the model studies afford the only quantitative data available and will be used with a full appreciation of the limitations and probable inaccuracy. The tests have, in general, shown the inefficiency of a spillway structure of the type tested. The studies clearly point out the waste of material, and the possible danger to the structure and property below the dam if improperly designed side channel spillways are used. If contraction of the collection channel is necessary because of physical limitations, the reduction in area should be compensated for by an increased slope. The greater velocities created by the steeper slope must be then dissipated. Erosion could be

lessened or prevented by extending the paved discharge channel down stream with facilities for inducing a hydraulic jump on the apron.—*H. E. Babbitt.*

**Using the Engineer's Transit to Determine Orifice Coefficients.** THOMAS D. SMITH. Eng. News-Rec. 118: 717 (1937). Method of determining contraction coefficients for standard orifice with aid of engineer's transit, as used in hydraulic laboratory of University of Delaware, is described. Customary method, calipering contracted section, is not entirely satisfactory.—*R. E. Thompson.*

## STREAM POLLUTION AND CONTROL

**Lake Drinking Water Hazards during the Summer Season—the Case of Squam Lake.** ANON. New Hampshire Health News, 15: 8 (1937). Members of family stopping at tourist camp on Squam Lake were taken violently ill, presumably from drinking water piped from the lake, which is extensively used for bathing. A survey by State Board of Health in 1933 had shown that more than 60 cottages, representing more than 30% of total, maintained such lake intakes. More than 25% of water samples collected during survey from different parts of lake disclosed presence of B. coli in 10 c.c. portions. Board is powerless to compel cottagers to use other sources of drinking water. However, in case of any lake or pond, the water of which may be used during summer for drinking or culinary purposes by any hotel, tourist home, public eating place, juvenile or tourist camp, the board requires that such water shall be effectively chlorinated, if permitted to be so used at all.—*G. C. Houser.*

**Pollution of the Mokelumne River Winery Wastes.** PAUL A. SHAW. Sew. Wks. J. 9: 599 (1937). The Mokelumne River, flowing through the grape-producing San Joaquin County in California, receives the liquid wastes from five wineries. Their discharge of still slop alone, resulting from the production of brandy used in wine fortification, produces a pollution load in the river of an average population equivalent of 225 persons per ton of grapes utilized. The mixed wastes not only produce a high oxygen demand but form an excellent medium for the growth of various fungi, which, in turn, cause a high mortality to fish life. In a study of the effects of this pollution, it was noted that the minimum dissolved oxygen, prior to the wine producing season, varied between 87% and 96% saturation, and that during the winery season, the minimum, above the first source of pollution, was 92%. Because of this pollution, these values fell to a minimum of 60% downstream to a dam, which, as a source of reaeration, raised this value to 88%. Below this the saturation again fell to a minimum of 46%.—*Arthur P. Miller.*

**Effect of Drought Conditions in Ontario on Water Supplies and Disposal of Sewage.** A. E. BERRY. Eng. Cont. Rec. 50: 81; 11 (1937). Data are given regarding the drought during summer of 1936, when rainfall was only 81% of normal. New peaks in water consumption caused serious difficulties and discharge of sewage effluents into streams with much reduced flows, coupled with prevailing high temperatures, resulted in decomposition and the production of offensive odors. No disease outbreaks occurred.—*R. E. Thompson.*

**Pollution of River Raisin by Beet-Sugar Factory Wastes.** W. D. LOREAUX. W. Wks. and Sew. **84:** 335 (1937). River Raisin is polluted by beet sugar wastes from a factory at Blissfield, Mich. Polluting wastes are (a) flume water, (b) process water, (c) lime cake water, which three are discharged during sugar campaign lasting 90 days, and (d) Steffen's waste, which is impounded for some time to permit some biological improvement before discharging into the watercourse. The problem has been studied by Michigan authorities and by the industry, and from these studies arise the following suggestions for reducing the pollution: (1) Treatment of flume and process waters, (2) handling lime cake in dry state and disposal as fertilizer, and (3) spray drying of part of Steffen's waste by utilization of waste heat at the industry with prolonged detention of remainder of waste to increase biologic stabilization. Steffen's waste offers value as a fertilizer for alkali-tolerant crops such as sugar beets. Author describes experiments on this subject.—*H. E. Hudson, Jr.*

**New Pollution Bill Passed by Senate.** ANON. Eng. News-Rec. **119:** 293 (1937). On August 16th, Senate passed House-approved measure with certain additions. Net effect of amended bill is to: (1) Create in Public Health Service a division of water pollution control. (2) Direct this division, in coöperation with Corps of Engineers and state and local authorities, to prepare comprehensive plans for reducing pollution in navigable waters and tributaries, and to promote enactment of uniform state control laws and interstate compacts. (3) Authorize grants-in-aid by federal government to local governments and industries for construction of treatment plants. (4) Set up board of 5 engineers in Public Health Service to divide country into sanitary districts, establish minimum pollution standards in each and review plans submitted in applying for federal grants. (5) Declare that discharge of waste into navigable waters in violation of board's regulations constitutes public nuisance, which may be proceeded against by federal injunction proceedings after expiration of 3 years from date of enactment, without nullifying right of parties injured by pollution to sue.—*R. E. Thompson.*

**Pollution Control in Pennsylvania.** ANON. Eng. News-Rec. **119:** 47, 76 (1937). Declaring that discharge of sewage and industrial wastes into waters of state is not natural use of these waters, bill recently signed by governor forbids pollution of waters of commonwealth, except as permitted by Sanitary Water Board of Dept. of Health. Pollution is defined to include introduction of any substance which will render water unclean to extent of being harmful to public health or to animal or aquatic life, or will be detrimental to use of water for domestic water supply, for industrial use, or for recreation. Board may order discontinuance of discharge of sewage at any time without regard to any previous permit. Municipality directed by board to cease pollution must construct treatment plants of design satisfactory to board. If money for same is not available and if bonds could not be sold without exceeding debt limit, or if voters refuse to approve bond issue, municipality must issue non-debt revenue bonds secured by sewer rentals. Board, at its discretion, may issue revokable permits to discharge sewage, except for discharge into water not

now receiving sewage or industrial wastes. All persons now discharging industrial wastes into water must file notice of same within 90 days of passage of act. Drainage from operating mines is exempt until practical means of treatment becomes known. Board is authorized to make complete pollution survey of waters of state and to conduct research on treatment of industrial wastes.—*R. E. Thompson.*

## WATER SUPPLY—GENERAL

**The Water Supply of London.** G. F. STRINGER. The Eng., **164**: 59 (1937). The history of the London water supply is one full of interest and romantic episodes. In the thirteenth century they had in every street and lane of the city "divers fair wells and springs." In 1236 Henry III granted to the citizens and their successors liberty to convey water from the village of Tybourne by pipes of lead into the City. In 1582 Peter Morrys was given a 500-year lease for the right to pump water from the Thames by a pump placed on London Bridge. A significant step leading towards the present status was taken early in the seventeenth century by the creation of the New River Company. This was followed by the creation of the Shadewell Waterworks in 1669, the Ravensbourne Waterworks in 1701, the Southwall and Vauxhall Co. in 1771, the Lambeth Waterworks Co. in 1785, the Grand Junction Waterworks Co. in 1798, and the West Middlesex Water Co. in 1808. The following century was marked by competition and amalgamation, culminating in 1902 by the creation of the present Metropolitan Water Board.—*H. E. Babbitt.*

**Liverpool Waterworks. Supplies During 1936–'37.** ANON. Surveyor. **92**: 194 (1937). A résumé of the annual report for the year ending March 31. The avg. daily demand was 44,493,000 Imp. gal. which is an increase of 1.26% over last year, and the greatest rate of demand on record. The increase is possibly a reaction from the requests for economy on the part of consumers during the drought years of 1933 and 1934.—*H. E. Babbitt.*

**Chemical Water Purification in Switzerland.** H. GUBELMANN. Schweiz. Verein von Gas- und Wasserfachmännern Monatsbulletin **17**: 6, 121 (1937). Perspective of water supplies and broadening of chemical treatment methods is given. Due to favorable geological conditions water demand till now is exclusively secured from underground sources and to a minor extent from deep lakes. In 1934 water consumption was 52,800 m.g.; out of this amount 25,344 m.g. were treated, 19,000 m.g. chlorinated. With absence of considerable amount of organic matter small dose of approximately 0.1 p.p.m. chlorine is sufficient to complete sterilization. In the city of Bern at times a chlorophenol taste appeared which was not always removed by preliminary treatment with potassium permanganate; since 1934 preammoniation proved satisfactory and economical. To prevent ground water from deteriorating and to secure efficiency of chlorination at increasing demands pollution of water resources is to be carefully controlled. To meet higher standards of purity broadening of simple chlorination methods is desirable; for small supplies Katadyn-sterilization after careful examination seems advisable.—*Manz.*

**Automatic Chlorination of Two Water Supplies for the City of Saint John.** G. G. Hare. Eng. Cont. Rec. 50: 68: 33 (1937). Water supply of St. John, N. B., city of 65,000, is derived from 2 chains of lakes situated in opposite directions from city and some 12 miles apart. Loch Lomond system, series of 5 lakes, serves section on east side of St. John River and Spruce Lake system, group of 3 lakes, serves section on west. Although supplies are of excellent natural quality, chlorination was adopted in 1936 as precautionary measure. Equipment is of automatic type, hydraulic turbines being installed to inject the chlorine into supply mains, which carry pressures of 55 and 5 pounds, respectively.—*R. E. Thompson.*

**Colorado River Water for California.** Julian Hinds. Civ. Eng. 7: 573 (1937). Los Angeles is surrounded by a fertile basin occupying about 1.4% of the area of the state. Within this area are to be found about half the wealth and population of the state—and about 1% of its water resources. Thus it is easy to see why water is at a premium. Except at times of flood every drop is saved for use. Conservation is accomplished largely by underground storage. The nearest important water source lies 200 to 300 miles to the north in the snow-capped high Sierras. A careful survey of all possible outside sources reveals that there is only one additional source to which to turn—the Colorado River, 300 miles away, across an inhospitable area of mountains and desert. Water for the system is to be released from Lake Mead into the river channel and rediverted at Parker Dam, 155 miles downstream. The main aqueduct consists of a series of concrete-lined tunnels, concrete pipe lines, buried concrete flow-line conduits, and concrete lined canals with a total length of 242 miles. The line is designed for a peak discharge of 1,605 c.f.s. with an average of 1,500 c.f.s. The water level above Parker Dam is to be 450′ above sea level whereas parts of the area to be served lie 550′ higher. Also fall must be provided to maintain flow in the aqueduct. Hence the water must be pumped. There will be five pumping stations with lifts varying between 144′ and 441′. The ultimate power consumption will be 344,000 hp. provided from the power plant at Boulder Dam. The work on the aqueduct has resulted in many important improvements in construction equipment and procedure. In the tunnels the old heading-and-bench procedure has been almost entirely replaced by automatic feed drills mounted on jumbos. It is estimated that the initial development will cost $220,000,000 and will be completed in 1939. It is now about two-thirds complete.—*H. E. Babbitt.*

**Inland Water Survey of Great Britain.** R. G. Hetherington. The Engr., 164: 53 (1937). British rivers tend to be short and small, and serve a dense, industrial population. As a result the Government set up a committee "to advise on the inland water supply for Great Britain on the progress of the measures undertaken and on further measures required, and, in particular to make an annual report on the subject." The information that is needed relates to rainfall, surface water and underground water. In the case of the first two, standards for their measurement have been defined in governmental publications. As regards underground water nothing has yet been published. The conversion of all records into standard forms of statement and their

publication will relieve the task of interpreting the records. The cost of the survey will be borne in part by the Governmental Departments affected supplemented by occasional grants through the Ministry of Agriculture and Fisheries.—*H. E. Babbitt.*

**London's Water Supply. The Kempton Park Pumping Station.** ANON. Surveyor. **92**: 106 (1937). The works are for the purpose of filtering water from reservoirs supplied directly from the Thames, and then pumping it to other storage reservoirs. The total capacity of the filters is 36 million Imp. gal. per day. The 24 primary filters are followed by 12 units of slow sand filters. There are two engine houses. One contains six large steam engines, two of which are used to pump the unfiltered water. The other engines are for pumping filtered water. In the other building are two marine engines and two steam turbine driven pumps. There are three 48-in., one 42-in., and one 36-in. filtered water mains leading from the station. Between 36 and 40 million Imp. gal. are filtered daily and about 50 million gal. are pumped.—*H. E. Babbitt.*

**Forecasting Mountain Water Supply by Photographing Snowfall.** D. D. GROSS. Eng. News-Rec. **119**: 310 (1937). Large part of Denver's water supply is obtained from melting snow above Continental Divide of Rocky Mountains. During past few years, H. L. Potts has been developing original method of forecasting runoff from accumulated snow. At regular times, photographs are made from same spot which take in panorama of many miles of Divide, including rugged peaks more than 14,000′ high. Snow that falls on peaks is driven by wind into numerous gulches and since each spot of white on photograph represents bank of snow which has filled or partially filled a gulch, it follows that the larger the area the deeper the snow. Percentage of snow area is determined and compared with runoff, allowing for precipitation as rain in late spring and summer. Method is believed to be most rapid yet developed and makes possible consideration of snow accumulated in places inaccessible by other methods. Results thus far have been very gratifying.—*R. E. Thompson.*

**Annual Report of Little Rock, Arkansas, Municipal Water Works. Dec. 31, 1936.** This is the first annual report of the new water works dept. of Little Rock, Ark., reporting on operation of property acquired April 1, 1936. Water is supplied to the City of Little Rock, pop. est'd. at 82,070, consumers 16,617, and wholesale to the City of North Little Rock, pop. 19,900. Report contains statistics of various departments of such works and is submitted by L. A. Jackson, Manager. Income for 9 mo., April to Dec. inc.—$465,166.23. Cost of supplying water per m.g. equalled $57.00 per m.g. for total direct expenses, $67.39 for total operating expenses; amount received $241.00 per m.g. Ave. rainfall for yr. 34.88″, a deficiency of 12.66″ from normal. Total length of pipe in system including 1″ to 24″—224.664 mi., an increase of 0.787 mi. since April 1; data is given in report of mileage of pipe of various sizes. System contains 958 fire hydrants, 1558 valves 2″ to 24″, is 100% metered and has 16,712 services including 95 fire services. Ave. daily pumpage was 6.69 m.g., total pumpage for the yr. 2449.7 m.g.; ave. per capita consumption for Little

Rock itself, 70 g. per day; North Little Rock 47 g.p.d., both cities combined 64 g.p.d. Water is obtained from Arkansas River and wells, river used alone 218 days, river and well water used on 148 days, percent well water pumped being 12.2%. River water turbid and quite hard, mean turbidity being 700 p.p.m., with max. of 15,000 and min. of 10; ave. total hardness of 208 p.p.m., max. 298 p.p.m. and min. of 88 p.p.m. through softening with lime. Alkalinity of raw water, mean 108; plant effluent mean, 69 p.p.m. Water extremely high in chlorides, with mean of 425 p.p.m., max. of 1130, min. of 90. Total bacterial count on raw water, ave. 646 per cc., on effluent-3. No B. coli found in plant effluent. Complete information given on plant equipment. Report includes pictorial representation showing breakdown of income and expenditures, several pictures of the plant and works and 6 pages to new supply being developed on Alum Fork in Saline Co., approx. 33 mi. west of Little Rock. Water from this new supply to have total hardness of but 14 p.p.m. and total solids of 28.6 p.p.m., chlorides 4 p.p.m.—*Martin E. Flentje.*

**Report of Board of Water Supply, City and County of Honolulu. For Biennium Ending Dec. 31, 1936.** Sixth annual report (237 pp.) of board in charge of water works supplying city and county of Honolulu with estimated pop. 146,000. Contains report of Manager and Chief Engineer, Frederick Ohrt; is subdivided under headings: Water Supply (20 pp.), Operation and Maintenance (25 pp.), Water Sales (25 pp.), and Financial Statements (22 pp.). A supplementary section is also given of 90 pp. giving considerable hydrological data on both underground and surface supplies of territory. Annual total income for 1935 was $1,066,798.88; for 1936, $1,106,210.89, with no charges being made to the city and county for fire hydrants, either for rental or upkeep. Cost of supplying water per m.g. figured on total maintenance was $98.06 for 1935, $101.85 for 1936; cost figured on total maintenance plus fixed charges, $172.90 for 1935 and $181.47 for 1936. Rainfall recorded by 24 rain gages on highlands back of city. Rainfall at Lower Luakaha, Nuuana Valley was 123″ for 1935 and 143.63″ for 1936; 47 yr. ave. annual at this station 139.63″. Max. daily recorded rainfall, 1935-36 was 11.28″ on Feb. 27, 1935, compared with max. daily of 19.7″ on Nov. 18, 1930. Rainfall for 47 yr. period is tabulated. Total pipe in use, 1″ to 24″, 1935—275 mi., 1936—286 mi.; cost of pipe repairs per mi. 1935 was $49.78, in 1936 $57.61; in 1935 system had 282 mi. and in 1936 29.5 mi. of pipe less than 4″ in diam. Of the 1864 fire hydrants in use, 46 were installed in 1935 and 51 in 1936. Pressure range in system is from 15 to 175 lbs. 470 new services were added in 1935, 814 in 1936 to give a total in use at the end of 1936 of 19,572. Total meters in use equalled 18,881 (1935) and 19,574 (1936) with system 100% metered. Total water consumption for yr. was 6,958 and 7,091 m.g. with 83.0% and 84.7% passing through meters. Ave. daily consumption was 19.0 and 19.4 m.g.; per capita consumption 132 and 133 gal. per day per inhabitant, and 1010 and 990 g.p.d. per service. Water comes largely from deep wells (95%) driven to artesian areas directly below city, other portion of water from springs and tunnels. Water from artesian wells has been classed as excellent for practically all purposes, total hardness in 5 artesian areas varies from 43 to 144 p.p.m., chloride content from 42 to 232 p.p.m. Report includes a number of very thoroughly and well prepared

charts, graphs and tables which together with good descriptive matter makes the report interesting and informative.—*Martin E. Flentje.*

**70th Annual Report of the Commissioners of Water Works in the City of Erie, Pa., Year Ending December 31, 1936.** 75 pp. (1937). Operating and financial data given are, as usual, complete in every detail. Operations during 1936 resulted in net addition to surplus of $8,644.17. Value of water supplied to city without cost amounted to $61,909.46. Daily average consumption by estimated population of 120,000 was 24.82 mil. gal. and daily per capita consumption 206.81 and 122.66 gal., inclusive and exclusive of metered industrial and commercial use, respectively. Cost of collecting, purifying and pumping water (including depreciation) was $30.648 per mil. gal. Number of gallons of water pumped per pound of coal used was 389.68. Following average amounts of chemicals and wash water were used at Chestnut Street and West filter plants, respectively: alum, .218 and .240 g.p.g.; chlorine, 1.67 and 1.69 pounds per mil. gal.; ammonia, 0.98 pound (Chestnut St. only); wash water, 2.06 and 1.32% of water filtered. Analytical data given include monthly average bacterial count, turbidity, alkalinity and color of raw and filtered waters, average temperature and results of *B. coli* tests. Filtered water from both plants was consistently negative for *B. coli* in 10 and 1 cc. throughout year. Latter part of report includes brief description of works, detailed schedule of rates and tabulation of assessments for various classes of service. Wash water troughs at both plants were coated with zinc by new metallizing process, thus materially lengthening their life and obviating necessity of frequent painting.—*R. E. Thompson.*

**Fifty-fourth Annual Report of the Board of Water Commissioners of the City of St. Paul, Minnesota, Year 1935.** 85 pp. Extensive tabular and graphical data relating to the operation of the St. Paul water works during 1935 are given. The department again returned to consumers $90,678.61 in form of special 10% discount on water bills. Net operating profit amounted to $37,607.58 after payment of all fixed charges and interest. Only 214 shut-offs were made for non-payment of bills. Outstanding delinquent accounts total only $9,120.36. Two major emergencies occurred during year: approximately one-half of 90″ steel intake pipe which extends several hundred feet into Vadnais Lake suddenly floated to surface in November and shortly after 100′ section of concrete conduit from Mississippi River to Charles Lake without warning dropped vertically about 6′, probably due to faulty pile foundation. Sinking and anchoring of intake involved several weeks' work: conduit repair was effected by installation of 60″ steel siphon. Revenue per mil. gal. pumped, exclusive of frontage tax receipts, amounted to $117.10 and net income, including frontage tax receipts, $8.74 per mil. gal. pumped. Total cost of operation and maintenance was $51.99 per mil. gal. and total fixed charges $56.55. Detailed tabulations of analytical data, chemical, bacteriological and microscopical, are given showing the quality of the water as drawn from Mississippi River, after passing through the system of lakes, after filtration and during distribution. Colon group organisms were demonstrated in only 1 of the 2127 samples of plant effluent examined: average bacterial count on agar at 37°C.,

48 hours' incubation, was 7 per 1 cc. Positive *B. coli* tests in water from distribution system were traced to 2 main breaks. The bacterial data show evidence of aftergrowths in distribution system of bacteria growing on agar at 20° and 37°C., particularly during warmer months. It is noted that 20° bacteria decrease in number more rapidly during passage of the water through the lake system than the 37° bacteria. Following average data are given in regard to filtration and chemical treatment: wash water, 2.53%; length of filter run, 44 hours; alum used, 0.81 g.p.g.; chlorine and ammonium sulfate dosages, 0.75 and 0.16 p.p.m., respectively (latter expressed as ammonia); cost of chemicals per mil. gal., $1.98. Average daily consumption was 22.405 mil. gal. or 76.21 gal. per capita. Estimated population served is 292,000. System is 99.9% metered.—*R. E. Thompson.*

**Water Supply of St. Stephen, N. B., is Unique in Providing International Service.** A. A. Laflin. Eng. Cont. Rec. 50: 73; 9 (1937). St. Stephen and Milltown, N. B., and Calais, Me., all located on St. Croix River, were originally supplied with water from the river by the Maine Water Co. Owing to unpalatableness and quality of river water, well supply was developed in 1906–7 a few miles from St. Stephen, water for Milltown and Calais being sold to Calais Water and Power Co. No treatment is required. A reservoir is located at each end of supply main and old river pumping station in Calais is maintained in readiness for use in case of emergency, chlorine being applied at such times. Original well supply pumping station was equipped with duplicate gas producer plants, engines and triplex pumps. In 1919, electric motor was substituted for one of gas engines and in 1933, other engine was replaced with Diesel engine direct connected to centrifugal pump. Average daily pumpage is about 0.75 mil. gal. Town of St. Stephen is 100 percent metered.—*R. E. Thompson.*

**Historical Background and Important Features of the Montreal Water Works.** C. J. Des Baillets. Eng. Cont. Rec. 50: 68; 14 (1937). History of supply is reviewed in some detail and present works described and illustrated. Water is drawn from St. Lawrence River through 2 intakes and conducted by gravity through open canal to treatment works, where low-lift pumps elevate it to top of filters. Filter plant, which was designed for ultimate capacity of 300 m.g.d. and has present capacity of 150 m.g.d., consists of 48 rapid sand filters, 2 filtered water reservoirs of 5 and 20 mil. gal. capacity and automatic chlorination equipment. The filter under-drains consist of cast iron headers and laterals with $\frac{5}{16}''$ bronze nozzles and were designed for air-water washing. Filters contain 15″ of gravel in 4 sizes and 30″ of sand of 0.45–0.55 mm. effective size and uniformity coefficient of approximately 1.65. About 43% of water delivered by low-level pumping station, which is located at filter plant, is repumped at McTavish station to higher districts. Distribution reservoirs, of which there are 6, 4 being very small, have total capacity of 100 mil. gal. During recent years, consumption has varied between 116 and 164 m.g.d. In 1936, cost of operating filter plant was $2.17 and cost of operating low- and high-level pumping stations $5.70 and $6.59 per mil. Imp. gal., respectively.—*R. E. Thompson.*

**Water Supply Alternatives Confront City of San Diego.** ANON. Eng. News-Rec. 118: 908 (1937). Annual rainfall in San Diego varies widely and major runoffs, on which dependence for continuous water supply must be placed, may occur only at 10–11-year intervals. Thus, cyclic storage is necessary. In recent years, it has become apparent that even cyclic storage, with development of local sources to utmost, will suffice for only few years more. Only other source available is Colorado River, from which water could be secured in 2 ways: (1) by joining Metropolitan Water District of Southern California or (2) by bringing supply through All-American Canal and pumping it over mountains from Imperial Valley. Consulting water board (L. C. Hill, L. S. Ready and J. P. Buwalda) appointed last year have recommended latter scheme, capital cost being only $15,968,000 compared with $17,880,000 for former and ultimate capacity 100 and 75 m.g.d., respectively. In addition, latter scheme lends itself to step-by-step development and lower initial cost. Delivery system from Imperial Valley to reservoirs on coastal slope, distance of 42 miles, would include pumping lift and tunnel, present conditions indicating most economical plan to be pump lift of 2700′ and tunnel length of 7.33 miles. There would be 6 pumping stations, each working against head of 475′. On basis of ultimate development, water cost would be 7.85¢ per 1000 gal.—*R. E. Thompson.*

**Grand Lake Diversion Project.** ANON. Eng. News-Rec. 118: 732 (1937). Brief details are given of proposed Grand Lake trans-mountain diversion project of U. S. Bureau of Reclamation in Colorado, as described by Porter J. Preston. Purpose of project is annual diversion of 310,000 acre-feet of water from Western Slope for irrigation of fertile plains near Fort Collins on Eastern Slope of Continental Divide. Project would include construction of 3 large earth and rockfill dams, power plant, 13.1-mile tunnel and about 30 miles of canals. Cost is estimated at about $34,800,000.—*R. E. Thompson.*

## ADMINISTRATION AND PERSONNEL

**Public Contributions of Municipally Owned Electric Utilities Compared with Taxes of Private Utilities in Minnesota.** ARTHUR BORAK. Minnesota Municipalities, 22: 267–78 (Aug., 1937). Author reports a comparative study of contributions to municipal incomes by utilities, comparing the cash contributions and free services of publicly owned and operated utilities with the taxes paid by those under private ownership. Only 2 privately owned water plants now in Minn. The total municipal receipts were obtained and analyzed for 43 of 95 cities in the state; among them being the 3 cities of the first class, with populations of 50,000 and over; the 3 cities of the second class with pop. of 20 to 50,000; 4 of the 8 third class cities, pop. 10 to 20,000; and 33 of the 81 cities with a pop. less than 10,000. 3 tables are given summarizing (1) electric utility revenues transferred to other municipal funds and the value of free services in cities with municipally owned generating plants in Minn., the ratio of transfers and free services to gross revenue being given for the yrs. 1922 to 1934 inc., which ratios vary from 1.11 to 13.73% with an ave. of 6.27%; (2) taxes of several private electric companies in Minn., for the same yrs., showing ratios of total taxes to gross revenue varying from 7.50 to 13.75% with an ave.

of 9.65% and ratios of state and local taxes to gross revenue, min. 5.00, max. 10.51, ave. 7.43%; (3) ratios of contributions of public plants and taxes of private plants to gross revenues. A table is also given comparing the rates of the public and private utilities. Conclusions reached are that in Minn., the contributions of municipally owned electric generating plants generally have not been equivalent to the total of local and state taxes paid by privately owned utilities, public ownership has not resulted in comparatively low municiple tax rates or debt, and that electric rates have not been comparatively lowered.—*Martin E. Flentje.*

**Water-Works Operating Records for Progress and Protection.** R. L. DOBBIN. Am. City, 52: 8; 59 (1937). Regardless of their extent, records are of little value if improperly interpreted or merely filed. This article explains how various records in a water-works system may become valuable, both as a means of improving service and maintenance and of absolving the water works operator from blame for real or supposed deficiencies. Illustrations of the record forms used at the Peterborough, Ontario water works are included.—*Arthur P. Miller.*

**Appropriating Water Systems.** LEO T. PARKER. W. W. Eng., 90: 826 (1937). Review of recent higher court decisions with respect to the validity of state laws which authorize municipalities to appropriate privately owned water mains disclose that statutes are valid: (1) if the connection is a public necessity; (2) if reasonable compensation is allowed the owner of the water main; (3) if a definite time limit is fixed for appeal when the water main owner is dissatisfied with the allowed compensation; (4) if compensation is based on actual losses which will be sustained by the water main owner; (5) if a legal board is authorized by the municipality or the state to appropriate or connect with the privately owned main for public purposes; (6) if a reasonably correct description of the appropriated main was properly recorded. Various courts have held that the owners of property abutting public streets actually own the sidewalk to the center of the street. However, this does not mean that the owner of private property may without permission of the municipality install water pipes or mains in the streets.—*Lewis V. Carpenter.*

**What's Wrong in Waterworks Management?** E. GROSVENOR PLOWMAN. Eng. News-Rec. 119: 225-6 (1937). Municipal water utilities display 3 characteristic management weaknesses: (1) They contain complex and costly detours, such as city-wide legal counsel, accounting and purchasing. (2) They are often beset with tangled problems of over-functionalization, *e.g.*, duplication of records kept by other municipal departments in order to keep essential data in their own files. (3) They cannot, in general, withstand pressure of public opinion, even when it is wrong. Examples of these weaknesses in various city water departments are outlined. See following abstracts.—*R. E. Thompson.*

**Effective Waterworks Organization.** E. GROSVENOR PLOWMAN. Eng. News-Rec. 119: 269-70 (1937). Cf. previous abstract. In this article, P. discusses importance of sound basic organization as related to efficient man-

agement. Chief weakness in municipal water utility organization is apron-string relationship between regulatory branch of city government and utility, which can only be overcome by establishment of independent water board. P. considers systems of Erie, Pa., and utility operating in and around Oakland, Cal., to be best examples of management efficiency. These organizations and their accomplishments are discussed. Sound basic organization is the necessary first step toward permanent and efficient management. See following abstract.—*R. E. Thompson.*

**Improving Waterworks Management.** E. GROSVENOR PLOWMAN. Eng. News-Rec. **119:** 322-4 (1937). Cf. previous abstracts. From numerous examples available, some of best operating practices of municipal water utilities are reviewed and general suggestions are made relating to improvement in organization and management. Elements of sound systems of personnel administration, purchasing, engineering and minor construction, plant operation, etc., are discussed, together with proper relationship between the public's representatives and the salaried executives, it being pointed out in regard to public relations that both the latter groups have a part to play. Water utility executives should be conscious of their ethical responsibility for the public welfare.—*R. E. Thompson.*

## TUNNELS, AQUEDUCTS, ETC.

**Wood Stave Pipe.** ANON. Civil Eng. (British), **32:** 55 (1937). Woods mainly used are California redwood, British Columbia Douglas fir, or Columbia pine. The redwood should not be creosoted, as it is almost ideal for the purpose without treatment. The useful life of redwood pipe is 50 years but there are many constructed 30 or 40 years ago which show no signs of decay. The material is not limited to any climate or soil, it will stand up to extreme heat or excessive cold, the dryness of the desert and the dampness of the tropics. The thickness of the staves is determined, not by the internal bursting pressure, but by the required thickness of the shell to resist any tendency to go out of round. The width of the stave is of minor importance. The whole of the stress due to internal pressure is taken by the steel bands, formulas for the diameter and spacing of which are presented. The expressions are empirical. The steel bands are usually made of rods of mild steel with 55,000 to 65,000 lb./sq. in. tensile strength with an elastic limit of about $\frac{1}{2}$ of the ultimate limit. Figures show wood pipe laid in position indicating ideal construction for pipe either over or underground.—*H. E. Babbitt.*

**Use of Safety Primers in Tunneling.** R. E. MUNN. Eng. News-Rec. **119:** 158 (1937). Extensive use of safety primers in Colorado River aqueduct tunnels for water supply of Los Angeles has established procedure suitable for general adoption, which is described. Briefly stated, safety primer consists of electric blasting cap inserted and glued into longitudinal hole in wooden plug.—*R. E. Thompson.*

**Fast Tunneling Clinches Sale of Water.** ANON. Eng. News-Rec. **119:** 59 (1937). Tunnel, 18.6 miles long, was recently completed in 7 months, in-

creasing water supply of Charleston, S. C., by 50 m.g.d. (which can be boosted to 80 m.g.d. by pumping). In 1917, principal elements of water supply system were Goose Creek Reservoir and Hanahan pumping station about 12 miles north of city. Canal-and-pipe aqueduct west from Goose Creek Reservoir to tap Ashley River, which was dammed at Bacon's Bridge, and pumping station were completed in 1919, but were not used until 1927, when great drought made it necessary. The drought also proved Ashley River to be a very unsure source of supply and it was decided to build 4.5-mile tunnel aqueduct from Edisto River to nearest branch of Ashley River about 10 miles above Bacon's Bridge Dam, water thus secured to be conveyed to Goose Creek Reservoir through existing aqueduct. New tunnel is continuation of old one and conveys water direct to Goose Creek Reservoir. Construction was prompted by proposal of pulp and paper company to locate new mill in city if latter would guarantee water supply. Contract for minimum of 25 m.g.d. was signed prior to starting tunnel. Ground through which tunnel runs consists of mixed material overlying bed of marl, virtually a soft calcareous rock containing about 17% moisture. Examination of old tunnel, which was unlined, showed that almost no deterioration had occurred during 8 years' service and that surface of marl seemed to have hardened; new tunnel, therefore, was left unlined. Same section (7-foot diameter modified horseshoe of about 40.5-square foot area) as old tunnel was adopted. The fall in the 23.5 miles from Edisto Creek is 19 feet. Cost was less than $1,000,000. (See following abstract).—*R. E. Thompson.*

**Marl Tunneling Methods.** J. E. Gibson. Eng. News-Rec. **119**: 60 (1937). Methods employed in constructing tunnel described in previous abstract are outlined. In places, marl was quite hard, approaching hardness of limestone, and having calcium carbonate content of probably as high as 88%; at other points it was much softer, being readily cut with knife or chisel and having calcium carbonate content as low as 45%. Construction was carried on from 17 shafts and 2 portals. Tunneling was done by blasting, larger pieces being broken up and section trimmed to within few inches of full size with pneumatic spades. Final trimming was done by hand, using carpenter's adz, smooth-surfaced uniform section being thus obtained. The 98,000′ of tunnel was driven, including sinking of shafts, in 9 months. Actual tunneling time was a little over 7 months.—*R. E. Thompson.*

**Sea Water Intake for Water Softening Plant, South Carolina.** Anon. Eng. News-Rec. **118**: 926 (1937). Unit prices given from 3 lowest bids on cast iron intake for delivering sea water for regeneration of zeolite units of new softening plant at Sullivan's Island.—*R. E. Thompson.*

**The Crossing of Treacherous John's Pass, Fla.** S. K. Keller. W. Wks. and Sew. **84**: 209 (1937). Description of installation of 10″ c. i. pipe across an 800′ tidal inlet. Strong currents made the work difficult. Ball and socket joints were used and the pipe was anchored to the channel bottom by heavy H-beams. Line was floated out from one shore to other.—*H. E. Hudson, Jr.*

## IMPOUNDING RESERVOIRS

**Precasting Record-Size Pipe.** Eng. News-Rec., **116:** 852 (1935). Details given of casting and curing of record-size reinforced concrete pipe for 9.6-mile section of main pipe line in distribution system of Colorado River aqueduct from Cajalco Reservoir to Pasadena's Pine Canyon Reservoir. Inside diameter will be 12 feet 8 inches and average head 55 or 60 feet (maximum 80). Theoretical capacity of line is 750 second-feet. Pipe is being cast in 12-foot sections. Joints are designed for 2 separate cement-mortar fillings; one being poured and rodded as soon as pipe is in place in trench (18 to 28 feet deep) through orifice at top, the other, filled from inside of pipe by pointing, only after backfilling has been completed. Laying of pipe is followed by pouring of backfill cradle, which supports lower portion of each section.—*R. E. Thompson.*

**The Silting of Reservoirs.** ANON. Eng. **144:** 233 (1937). Laboratory experiments show that if turbid water is admitted to the head of a model reservoir, a cloud of suspended matter progresses in close contact with the bed until the weir is approached. The cloud is then observed to make its way along the upstream face of the dam and to pass over the crest. Such behavior is confirmed by observations in lakes and reservoirs, and it is evident that any system of scouring or under sluices will act advantageously towards keeping down silting. The magnitude of their effect is affected by various variables. These variables include the amount of silt and the rate of flow of the river, the nature of the silt, its rate of deposition, its density when deposited, etc. The problem of mitigating the silt evil in reservoirs is complicated by their capacity and by the nature of the run-off. Troubles from silting have been encountered particularly in the Union of South Africa and studies of silting have been made there and at Lake Mead in the U. S.—*H. E. Babbitt.*

**Alluvial Deposits in Reservoirs, Their Importance and the Means to Lessen or Prevent Them.** MARCO VISENTINI. Special Supplement, Large Dams, Wtr. and Wtr. Eng., **10:** 13, (Sept. 1937). From paper presented to 2nd. World Congress at Washington, D. C., Sept. 7–12, 1936; being a compendium of what is positively known up to date in Italy about alluvial deposits in reservoirs, and the transport of solid material in the water courses. Data is given for various reservoirs, and one built on the rapids of Torre (Province of Udine), Dam of Crosis, was built in 1896 and had a capacity of 150,000 cu. m. and a catchment area of 63 sq. km. is at the present time completely filled with alluvial deposits. This has taken place in 12½ yrs. Ave. yearly rainfall is 1800 m.m. Yearly solid contribution to the basin is about 190 cu. m. per sq. km. Similar data is given for other reservoirs, among them the res. of Monreale on the river Cellina, Lake of Alleghe; res. at Serra on the Cismon, etc. The figure for 12 res. only are tabulated whereas 140 or more important res. are known in Italy. The yearly contribution in cu. m. per sq. km. varies from a min. of 28 to a max. of 1944, with a majority of the values between 400 and 900. Some streams do not carry deposits, others must be stopped by reforestation and some by strengthening the banks to avoid side erosion, and the construc-

tion of cross works in the bed of the stream above the res. Alluvial deposits have been found difficult to flush out of a res.—*Martin E. Flentje.*

## TREATMENT—GENERAL

**Water Purification and the Public Health.** JOHN H. GREGORY. Civ. Eng. 7:621 (1937). Of the several branches of sanitary engineering those of greatest importance are water supply and sewage disposal. It is here intended to review very briefly some of the broader aspects of what has been accomplished in water purification, primarily in this country, since the beginning of the twentieth century. A satisfactory water is clear and colorless, free from objectionable tastes and odors, of satisfactory chemical content, soft, and of the highest hygienic quality. Water supplies in general may be classed as surface supplies and ground-water supplies. Surface waters almost always have turbidity, the removal of which by storage and sedimentation alone is seldom sufficient. In general, filtration preceded by chemical treatment will be required for its removal. Color can be reduced by the bleaching action of the sun, and also by the adsorptive power of aluminum sulphate. Some tastes and odors can be removed, or reduced, by aeration, but aeration may not be a sufficient corrective. Waters containing free carbon dioxide are corrosive in a degree dependent upon the amount of carbon dioxide present. Hardness is generally due to the presence of bicarbonates or sulfates of calcium and magnesium. The zeolite process of water softening is now coming more and more to the front, sometimes in combination with the lime and soda-ash treatment. Iron in solution in a water may be objectionable if present in excess of a certain percentage. Correction lies in aeration, generally followed by filtration. This procedure may also be used in the removal of manganese. Correction of a water from the bacteriological standpoint is, of course, of the greatest importance. This may be done by storage, disinfection, filtration, or by a combination of these processes. Two types of filters are in general use, the so-called slow sand filter and the rapid sand filter. Disinfection, or killing of bacteria by means of a chemical, may be used solely, or as a further safeguard when water is filtered. In addition to the striking reduction in typhoid fever rates, following water purification, there has been a substantial reduction in other diseases which can be traced to the treatment of public water supplies.—*H. E. Babbitt.*

**Hydrogen Ion Control in Water Purification.** C. H. S. TUPHOLME. The Surveyor, 91: 561 (1937). The most efficient coagulation takes place at a definite pH value, this value being different for different waters, varying from 4.2 to 7.6 when alum is used as a coagulant. The optimum may change with the season, temperature, change of source of supply, etc. The residual alkalinity does not usually give any definite idea of the method of treatment. Poor coagulation may be caused by an over as well as by an underdose of coagulant. It is not always economical to add sufficient alum to obtain the best possible floc. Sufficient quantities may be added to obtain a fair floc which will settle satisfactorily in the time that the water is allowed to stand in the coagulation basin. In some waters the pH range for a satisfactory floc may be narrow, in others rather wide. Where ferrous sulphate is

used a higher alkalinity is required; in fact, it is essential to have a pH of 8.8 or above for the most efficient results in most waters. pH control is just as important where sodium aluminate is used either alone or with alum. In many cases it is necessary to add alkali after coagulation to prevent corrosion of the distribution system. Proper control at this point can be obtained only by pH determination. The water should be adjusted to a pH which gives the maximum precipitation within several hours, and then approximately to the saturation equilibrium after filtration. By using both alkalinity and pH tests one can tell almost immediately the most desirable treatment for a given water. When zeolites are employed it has been found that the pH of the brine solution has a marked effect on the efficiency of the revivification of the zeolites. With alkaline waters above 7.8 behavior of the zeolite is erratic, while with acid waters there is an apparent leaching of the zeolite. pH control is obviously beneficial. An acid condition should always be avoided in swimming pool water which frequently results where alum is used as a coagulant. A pH of 7.2 to 7.6 is regarded as satisfactory. The residual chlorine in swimming pool water should never exceed 0.5 p.p.m.—*H. E. Babbitt.*

**Experimental Studies of Natural Purification in Polluted Waters. X.—Reoxygenation of Polluted Waters by Microscopic Algae.** W. C. PURDY. Public Health Reports, **52:** 29, 945 (1937). Minute cells of green algae, sufficiently numerous to tint the water a scarcely perceptible green, and under average daily conditions as to sunlight, produced measurable amounts of dissolved oxygen (D. O.). The medium used simulated heavily polluted water, in that it contained sufficient dissolved organic matter to sustain a bacterial content which, in various separate experiments, reached maxima varying from 5 millions to 13 millions per cc. However, only one kind of bacterium was present, and there were no protozoa, predatory or otherwise. All cultures (excepting one group) were in completely filled bottles. Atmospheric aeration was eliminated. Duration of the experiments varied from 10 to 17 days. Cultures containing bacteria but no alga cells showed serious decrease in D. O., this sometimes being entirely depleted. Cultures containing the minute alga in addition to the bacteria showed similar decrease of initial D. O., but a program of recovery and replacement soon developed, becoming noticeable about the third day, when D. O. began to increase, coincident with increase in number of alga cells. This oxygen always prevented depletion by a wide margin, and sometimes produced supersaturation. Cultures containing no bacteria, but alga cells only, showed no decrease of initial D. O., but a progressive increase, corresponding in general with the daily increase in alga cells. Check cultures in darkness in the 20°C. incubator showed (a) growth of bacteria similar to that in the cultures exposed to light, (b) no increase, or slight increase, of alga cells, (c) no replacement of D. O. Cultures having contact with the air show results very different from those shown by sealed cultures. The drop in initial oxygen is about one-third as great, then remains relatively stable, neither becoming depleted nor regaining an excess from algal activity. The alga-made oxygen as shown by titration is only about one-tenth the amount obtained from similar quantities of alga in the sealed cultures meantime. Apparently, exposure to the air, as obtains under conditions in

nature, permits escape of most of the oxygen. Includes general items of set-up, 20 tables, 2 charts.—*R. E. Noble.*

**Better Water Treatment.** Paul Hansen. Eng. News-Rec. 118: 801 (1937). Developments in water purification are reviewed. Improved coagulation has resulted from perfection of mixing and more complete solution of the coagulant. A primary development is the use of sodium silicate in conjunction with alum. More attention is being given to the use of iron compounds. In field of filtration, there is tendency to employ coarser sand. Surface washing in conjunction with usual backwash has been adopted at several plants and consideration is being given to use of porous materials for filter underdrains. There is growing realization that storage for filtered water is important part of purification plant. Ease of applying powdered activated carbon has resulted in its almost universal use. Much higher dosages are being employed. Granular carbon units have been installed at several plants. Lime softening has been improved at Springfield, Ill., by admitting treated water to bottom of sedimentation basin in such manner as to cause its rise upward through cloud of precipitated calcium carbonate, which promotes precipitation and clarification. Use of pyrolusite for manganese removal will probably extend at plants where softening is not employed.—*R. E. Thompson.*

**Functions and Value of a Control Laboratory in Connection with the Purification of Water.** Norman J. Howard. Eng. Cont. Rec. 50: 68; 24 (1937). A discussion of the function of a water works laboratory in determining the treatment required, controlling tastes and odors and adjusting operating practice to ensure production of water of the most desirable quality.—*R. E. Thompson.*

**Distilled and Sterile Water Supply in a London Hospital.** Anon. The Engineer (British), 163: 302 (1937). A central automatic plant for the distillation and sterilization of water working on the Jessop-Cook system at the University College Hospital, London provides a supply of hot and cold distilled and sterile water at draw-off taps in the operating theatres, sterilizing rooms, and elsewhere.—*H. E. Babbitt.*

**Preparation of Plans for Water Works Structures.** Anon. Public Works, 68: 9: 16 (1937). Check list is given for items affecting design of water treatment plant together with information required by several states when plans are submitted to Health Dept. for approval.—*Martin E. Flentje.*

**Water Supply and Sanitation.** Samuel A. Greeley. Eng. News-Rec. 118: 631 (1937). General review and discussion of advances in fields of water supply and disposal of sewage and other wastes. The demands of the public in regard to water quality are ever becoming more exacting.—*R. E. Thompson.*

**Recent Developments in Air Diffusion for Sewage and Water Treatment.** Frank C. Roe. Eng. Cont. Rec. 50: 68; 54 (1937). Diffusers of porous plate

type only are considered. Plates of much greater permeability are now being employed, frictional loss and susceptibility to clogging being thereby reduced. The size of bubbles released from high permeability diffusers is only slightly larger than those from relatively fine grades, owing to fact that bubble must approach a given size before its buoyancy overcomes the surface tension, permitting release. Experiments have shown that in multiple-plate installations of diffusers of 30 permeability or over, variation in permeability of individual plates should not be greater than 0.5. In low-permeability plates, uniformity of diffusion is noticeably affected by variation of 1.0 in rating. Rates of diffusion in excess of 1.5 cu. ft. per sq. ft. per min. are necessary to insure greatest uniformity of diffusion: if rate exceeds 3.0 cu. ft. per min., however, efficiency of oxygen absorption is impaired. For aiding flocculation, rate may be decreased to 1.0 cu. ft. per min. In installations whose primary function is agitation rather than oxygen absorption, rates of 4.0-6.0 cu. ft. per sq. ft. per min. are satisfactory. Aluminum plate holders have been developed which facilitate removal for servicing. Standard specifications for diffuser plates and tubes are included.—*R. E. Thompson.*

**The Treatment of Colored and Corrosive Water.** NORMAN J. HOWARD. Eng. Cont. Rec. **50:** 73; 13 (1937). General discussion of nature of color in water and its removal, relative suitability of various coagulants and adjustment of final pH value to prevent corrosion. Color should be reduced to not more than 20 and preferably to 15 p.p.m. Modern practice is to maintain pH value of delivered water at approximately 9.5.—*R. E. Thompson.*

**Progress in Water Works Practice.** NORMAN J. HOWARD. Eng. Cont. Rec. **50:** 83; 55 (1937). A general discussion, with particular reference to treatment and purification.—*R. E. Thompson.*

**Comparing Filtering Efficiencies by Determining Small Amounts of Coagulated Material in Effluents.** JOHN R. BAYLIS. W. Wks. and Sew. **84:** 310 (1937). Device for accurate measurement of exceedingly low turbidities consists of constant-rate cotton-plug filter attached to filter effluent. Cotton plug is removed and burned periodically, and ash content is taken as index of effluent clarity. Results may be read to 0.001 p.p.m. The instrument is valuable for comparing turbidities below the range of available turbidimeters, and gives a weighted average of effluent quality. Results for four filters over a 6-months period are given.—*H. E. Hudson, Jr.*

## CHEMICAL FEEDING, CONDITIONING AND SEDIMENTATION

**Singapore Water Treatment Experiments.** D. J. MURNAME. Surveyor. **92:** 228 (1937). Singapore waters always contain iron varying in quantity from 0.06 to 5.0 p.p.m. The iron is found partly in solution and partly in fine suspension as a constituent of a form of laterite. Dosing of the raw water with slaked lime on entering the slow sand filters has, in the past, effected a 75% removal of the iron. At Gunong Pulai rapid sand filter plant it was decided to change the sequence of operations by applying the slaked lime between the sedimentation tank and the filters. The sequence of operations then

became: aeration, application of powdered limestone, application of alum, mixing, coagulation and sedimentation, application of slaked lime, filtration, and chlorination. The pH value of the effluent from the sedimentation tank was approximately 6.0—the determining factor being the amount of $CO_2$ it contained; the dose of slaked lime was such as to bring its pH value to about 8.1. Increased filter runs resulted, iron removal at first declined but ultimately recovered to that experienced before the experiment was commenced, and the condition of the sand beds improved. Algal troubles have been experienced from time to time, especially with the slow sand filters. At the rapid gravity filter plant it was found that the use of the sedimentation tank following good mixing of the chemicals and coagulation solved the problem. It has been decided to convert one of the slow sand filters into a sedimentation tank and to dose the water entering it with limestone and alum and to dose the effluent from it with slaked lime to raise the pH to 8.1 in the hope of improving the results. The cost of filtration and chlorination has averaged 1.26 cents per 1,000 gal. (based on Straits dollars and Imperial gallons). The total water accounted for during the year amounted to 95.1% of the water delivered into the distribution system. This is claimed as a world's record for metering.—*H. E. Babbitt.*

**Experiences with Ferric Chloride at Emporia, Kansas.** S. F. Kunz. W. W. and Sew., **84**: 81 (1937). Adverse alkalinities and unsatisfactory flocculation forced use of ferric coagulant in order to produce an effluent free of residual coagulant. Ferric chloride lengthened filter runs, improved filtered water, reduced chlorine requirements, at a saving of $1.00 per M. G.—*H. E. Hudson, Jr.*

**Coagulation. III. Preparation of Silicate Solutions.** J. R. Baylis. W. Wks. and Sew. **84**: 221 (1937). Some difficulties have been experienced in preparing silicate solutions to aid coagulation. Some solutions form jellies, others give no aid to coagulation. The author gives a method for measuring silica concentration in solutions. Procedure for preparing useful silicate solution is as follows: untreated 42° Be. silicate solution having a $Na_2O$-$SiO_2$ ratio of about 1.0–3.2 is diluted to about 3% of $SiO_2$, then dilute sulfuric acid (N/1) is slowly added, accompanied by agitation, until methyl orange alkalinity reaches 1100 to 1300. Several trials may be necessary to reach this result. pH is valueless in estimating activity of prepared silicate solution. Solution of 1100 alkalinity may be more active in aiding coagulation than one of 1300 alkalinity, but the former is more likely to form a jelly than the latter. After preparing the solution and allowing it to remain standing for a few hours for development of coagulation-aiding power, the concentrated solution should be diluted to less than 0.6% $SiO_2$. Range of alkalinities of prepared solutions through which aid is given to coagulation is very narrow. Acidified solutions retard coagulation. Data and illustrations are given which show that small quantities of prepared silica aid, while larger concentrations retard coagulation shortly after preparation. After a day of development of activity, the range of concentrations in which silica aids coagulation is greatly broadened.

The form of silica which aids coagulation is believed to be a colloidal suspension of hydrous silica. Laboratory tests were confirmed by pilot-scale plant tests.—*H. E. Hudson, Jr.*

## SOFTENING AND IRON REMOVAL

**Coke-Carbon Dioxide Relations.** KENNETH G. SKINNER. Ind. Eng. Chem., **29**: 696 (1937). A quadrant chart is presented for graphically calculating weight of carbon dioxide obtainable, weight of coke required, or volume of flue gases when coke is burned for carbon dioxide production under various conditions of air supply, coke composition, temperature and pressure.—*Selma Gottlieb.*

**Water Softening and Its Prospect in Germany.** E. NAUMANN. Gas-u. Wasserfach. **80**: 7 (1937). The scarcity of fats in Germany brought up the suggestion that water should be softened to reduce the consumption of soaps and thereby save fats. The author compares different data published for this saving. He calls attention to the fact that only a small percentage of the output of waterworks is used in soap suds and that relatively few German plants furnish water hard enough to warrant softening (over 350 p.p.m. hardness). Although in such cases softening of the city supply may be economically justified, the country as a whole should get better results by promoting the use of water softeners in the home.—*Max Suter.*

**An Automatic Water Softening Plant.** W. AUSTIN SMITH. Public Works, **68**: 9: 9 (1937). Description is given of first zeolite water softening plant in Florida. Supply comes from 3 wells, 120′ deep, and water has hardness of approx. 17 g.p.g., largely calcium bicarbonate with about 2 gns. of magnesium hardness present and the same amount of non-carbonate hardness, iron content of 0.30 p.p.m. Zeolite process chosen because of lower first cost, no sludge disposal problem and lower operating and total costs. Completely automatic plant installed which required no added operators. Plant consists of 1–11′ diam. x 6′ steel coke aerator; 3–9′ diam. x 10′ vertical type zeolite units; 1–48″ diam. x 48″ brine measuring tank and 1 concrete wet salt storage basin. Zeolite employed is a high capacity, green-sand-base, zeolite. Capacity is 1.5 m.g.d., softening from 17 to 5 g.p.g. Practically only duty of operator is to see that salt is present in wet storage basin. Operation has been satisfactory.—*Martin E. Flentje.*

**The Use of Bleaching Earth for Softening of Brewing Water.** W. SCHAEFER AND M. THOMAS. Wochenschrift für Brauerei **54**: 25, 196 (1937). Investigations on softening of brewing water with bleaching clay with and without other chemicals. Montana Z, a Bavarian bleaching clay activated with mineral acids was used. After aerating with 0.5 percent clay for one hour partial softening takes place, magnesia being especially absorbed. After softening with lime or combination of other chemicals such as activated carbon, sodium phosphate, lactic acid, some reduction of residual hardness is observed. In all cases a considerable increase of silica content was noticed.—*Manz.*

**Softening Plant Improvements at Topeka.** DANIEL H. RUPP. W. Wks. and Sew. 84: 321 (1937). Kaw River water is flashy, hard, turbid, fairly highly polluted, and somewhat troublesome as to tastes and odors. The old treatment plant, in use since 1923, has recently been remodeled at a cost of $60,000. Plant now consists of the following steps: screens, low lift pumps, auxiliary well supply, aeration, grit removal, primary flash mix and flocculation with lime, soda, and alum, clarification with possible sludge return to primary mixer, secondary flash mixing with addition of ammonia, chlorine, alum, and carbon dioxide, secondary flocculation, secondary settling (plain sedimentation), addition of activated carbon, rapid sand filters, post chlorination and clear water storage, and high lift pumping. Numerous design details are given. Hardness is now reduced to less than 100 p.p.m. Savings in chemicals are about 13.4%. Chemical feed equipment is described.—*H. E. Hudson, Jr.*

## METERS

**Should Meters be Tested Singly or in Groups?** ARTHUR T. COOK. W. W. Eng., 90: 392 (1937). It makes no difference in the accuracy of the tests whether meters are tested singly or in groups. Group testing is more economical because a five point test requires almost one hour for completion. In labor the single test is from eight to ten times as expensive as the eight or ten group test, making the expense prohibitive. The limiting factor on the number of meters to be tested is the pressure of the supply. For 80 pounds incoming pressure 10 meters should be the maximum while for 100 pounds 12 is a good maximum.—*Lewis V. Carpenter.*

**Meter Testing Needs Improvement.** ARTHUR T. COOK. W. W. Eng., 90: 80 (1937). "Many companies are maintaining meter testing and repair shops which are actually detrimental to the interests of the company. The absolute abolishment of such shops and the purchase of new meters every two or three years would result in net financial gains for these companies." Author criticizes the reports that a good value accounted for water should be between 80 and 90%. Much of this charged up as leakage should really be charged to under registration of meters. The results of tests of a number of meters are shown. Emphasizes the necessity of having some one at the head of the meter department who knows his business.—*Lewis V. Carpenter.*

**New Metering Service for Louisville.** Eng. News-Rec., 117: 57 (1936). In 1935, 13,000 new meters were added to system of Louisville Water Company. bringing 48,600 consumers, or 75% of total, under metered service. Vaults for ⅝-inch meters were installed at cost of $6.66 each. Owing to adoption of stub system of accounting and continuous billing, re-routing of meter readers was necessary. Meter testing and repair department has been reorganized and enlarged to provide facilities for exchange, testing, and repair of each meter at intervals of 5 years or less. Although full effect of new meters was not evident in 1935, reduction of 5.5% in consumption was attributed to added meters.—*R. E. Thompson.*

**Maintenance of Water Meters.** CLEM A. GALLAGHER. W. Wks. and Sew. 84: 331 (1937). Present day meter neglect is amazing. Louisville, Ky., resets 75 meters per day, bringing in 75 meters for repair. Each is tagged. When meter is okay, half of tag is removed and filed, with record thereon of repairs. Other half of tag is returned after resetting meter and data on meter is noted on permanent meter record card. Procedure for testing and repair is given, also brief description of meter records with illustrations. Observations on tests for choice of meter are included, with recommendations on adequacy of present registration standards.—*H. E. Hudson, Jr.*

## WELLS AND GROUND WATER

**Well Boring. An Account of Present-day Practice: Middlesex Firm's Methods.** ANON. Surveyor. 92: 130 (1937). This is an account of an inspection of the works of LeGrand, Sutcliff, and Gell by a section of the Society of Engineers. Tools used in the making of bore holes were exhibited, together with descriptions and exhibitions of their actual operation.—*H. E. Babbitt.*

**Water Supply from the Lower Greensand.** ANON. Eng. 164: 200 (1937). Ample supplies of water are available in the lower greensand formation, but deep borings in this formation have, until comparatively recently presented considerable difficulties. Recently a 10″ well in this material has been developed to deliver 625,000 g.p.d. The well was constructed by the mud-flush system. The flow of 20,000 (British) g.p.d. is controlled by an "Arbon" valve at the surface in such a manner as to be free from sand.—*H. E. Babbitt.*

**Lower Greensand Water Bores.** ANON. The Eng., 164: 187 (1937). A new greensand bore yielding over half a million gallons (British) of soft, sand-free, water per day has been put down for Horlicks, Ltd. of Slough, using the mud-flush system of drilling. The 10″ hole, 1100′ deep, was put down in four months. Accompanying illustrations show a general view of the well-site, with derrick, and mud reservoirs, and the special "Arbon" valve fitted at the surface to enable the greensand water to be kept under control.—*H. E. Babbitt.*

**Lower Greensand Water Bores.** ANON. Surveyor. 92: 193 (1937). An account of recent successes with the mud-flush, rotary system; 500,000 Imp. gal. per day natural overflow. In the past deep borings to the greensand have always presented two difficulties: (1) the sudden rush of water and sand on encountering the water-bearing strata, and (2) the difficulty of procuring a constant flow of sand-free water. Now, with the mud-flush technique deep greensand borings may be drilled more rapidly and with greater chance of success. During drilling operations colloidal mud was kept in constant circulation through hollow drill rods and special drilling bits. The flow of water having once started from the well, is controlled by valves fixed to the well head.—*H. E. Babbitt.*

**Desert Water Tanks.** GLENTON G. SYKES. Eng. News-Rec. 119: 36 (1937). In more arid parts of the Southwest, especially in semi-desert regions

of southern Arizona, question of small but dependable water supply is of prime importance. Rainfall is seldom more than 5″-12″ yearly and over 50% of annual total falls during July, August and September. Dryness of climate and extreme heat for several months makes storage in open reservoirs almost impossible. Unique type of water bed or reservoir, called the "desert sand tank," was introduced more than century ago and is still in use. Essentially, it consists of dam or other impervious structure across streambed or large desert "sand wash," well bonded to bedrock and channel walls. Impounding basin formed soon fills with coarse sand and gravel, which is carried in abundance by desert streams owing to steep grades and heavy flows during rainy season. The water is stored in the voids of the material, which total 25-30% of volume. Evaporation losses are very much reduced and water is protected from contamination by animals and insects. Pore spaces are too large for capillary movement to properly develop, with result that upper layers of sand, when they dry out, act very much as an insulator. Water is drawn off either through perforated pipe in bed of basin or through shaft or collecting well against upstream face of dam.—*R. E. Thompson.*

**A New Deep Water Supply for Croydon.** ANON. The Eng., **164**: 132 (1937) A method of well boring used by the Layne-Well-System, Ltd. in America for oil wells and deep water wells has enabled a 17″ test hole to be drilled to a depth of over 1100′ in less than 300 working hours. A section of the subterranean formation encountered is shown diagrammatically. The strata encountered include, in order, top soil, chalk, upper greensand gault, lower greensand, and jurassic. The permeable strata are the chalk and the greensands. The first stage in boring the well was to make a test boring 8″ in dia. When the test boring has been completed to a depth exceeding that to which it is intended to drive the well, work is suspended for several days leaving the clay to settle in the hole. A screen is then set, the well lined, and then pumped sufficiently to determine its specific capacity and to obtain a sample of the water. The screen is then raised and the next stratum tested similarly. The work is carried on from the bottom upwards to ensure that any disturbance of natural conditions does not spread downwards to spoil existing work. The *first* test is to discover the permanent capacity of the well. When the permanent pump has been fitted the plant is operated for a specific period. It is subject to a guaranteed output covering a period of 12 months.—*H. E. Babbitt.*

**Sandy Wells are the Tough Ones!** MIKE WALTERS. W. Wks. and Sew. **84**: 314 (1937). Sandy wells cause excessive pump wear, hence it is well to develop a well and determine its capacity with temporary pumping equipment, later choosing proper pump. Pump wear is reduced and higher efficiency maintained by choice of wear-resistant parts.—*H. E. Hudson, Jr.*

**Transite Well Pipe and Screens.** WM. J. LUMBERT. W. Wks. and Sew. **84**: 206 (1937). To avoid corrosion, Scituate, Mass., used Transite well pipe and screens made of slotted transite pipe sections. Method of installation and details of construction are given. Wells are gravel packed.—*H. E. Hudson, Jr.*